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AMERICAN CONCRETE INSTITUTE

PROCEEDINGS

OF THE

TWENTY-FIFTH ANNUAL CONVENTION

Held at Detroit, Michigan

February 12, 13, and 14, 1929

VOLUME XXV

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TECHNICAL COMMITTEES

Technical committees of the Institute were discharged by action of the Board of Direction, February 14, 1929. Complete technical committee reorganization is still in progress as this book goes to press. Announcements of new committees will be made in issues of the American Concrete Institute *News Letter.*

BY-LAWS

AMERICAN CONCRETE INSTITUTE

Article I

MEMBERS

Section 1. Any person engaged in the construction or maintenance of work in which cement is used, or qualified by business relations or practical experience to cooperate in the purposes of the Institute, or engaged in the manufacture or sale of machinery or supplies for cement users, or a man who has attained eminence in the field of engineering, architecture or applied science, is eligible for membership.

Sec. 2. A firm or company shall be treated as a single member.

Sec. 3. Any member contributing annually twenty or more dollars in addition to the regular dues shall be designated and listed as a Contributing Member.

Sec. 4. Application for membership shall be made to the Secretary on a form prescribed by the Board of Direction. The Secretary shall submit monthly or oftener, if necessary, to each member of the Board of Direction for letter ballot a list of all applicants for membership on hand at the time, with a statement of the qualifications, and a two-thirds majority of the members of the Board shall be necessary to an election.

Applicants for membership shall be qualified upon notification of election by the Secretary by the payment of the annual dues, and unless these dues are paid within 60 days thereafter, the election shall become void. An extract of the By-Laws relating to dues shall accompany the notice of election.

Sec. 5. Resignations from membership must be presented in writing to the Secretary on or before the close of the fiscal year and shall be acceptable provided the dues are paid for that year.

Sec. 6. The Board of Direction may confer honorary memberships in recognition of services of an extraordinary meritorious character before the Institute. Honorary members shall be entitled to full membership privileges without the payment of dues.

Article II

OFFICERS

Sec. 1. The officers shall be the President, two Vice-Presidents, six Directors (one from each geographical district), the Secretary and the

Treasurer, who, with the five latest living Past-Presidents, who continue to be members, shall constitute the Board of Direction.

SEC. 2. The Board of Direction shall, from time to time, divide the territory occupied by the membership into six geographical districts, to be designated by numbers.

*SEC. 3. There shall be a Committee of five members on Nomination of Officers elected by letter ballot of the members of the Institute, which is to be canvassed by the Board of Direction *on or before September 1 of each year.*[1]

The Committee on Nomination of Officers shall select by letter ballot of its members, candidates for the various offices to become vacant at the next Annual Convention and report the result to the Board of Direction who shall transmit the same to the members of the Institute at least *60 days*[2] prior to the Annual Convention. Upon petition signed by at least ten members, additional nominations may be made within *20 days*[3] thereafter. The consent of all candidates must be obtained before nomination. The complete list of candidates thus nominated shall be submitted *30 days*[4] before the Annual Convention to the members of the Institute for letter-ballot, to be canvassed at 12 o'clock noon on the second day of the Convention and the result shall be announced the next day at a business session.

SEC. 4. The terms of office of the President, Secretary and Treasurer shall be one year; of the Vice-Presidents and the Directors, two years. Provided, however, that at the first election after the adoption of this By-Law, a President, one Vice-President, three Directors and a Treasurer shall be elected to serve for one year only, and one Vice-President and three Directors for two years; provided, also, that after the first election as President, one Vice-President, three Directors and a Treasurer shall be elected annually.

The term of each officer shall begin at the close of the Annual Convention at which such officer is elected, and shall continue for the period above named or until a successor is duly elected.

A vacancy in the office of President shall be filled by the senior Vice-President. A vacancy in the office of Vice-President shall be filled by the senior Director.

Seniority between persons holding similar offices shall be determined by priority of election to the office, and when these dates are the same, by priority of admission to membership; and when the latter dates are identical, the selection shall be made by lot. In case of the disability or neglect in the performance of his duty of any officer of the Institute, the Board of Direction shall have power to declare the office vacant. Vacan-

* Petition to amend Section 3 of Article II of these By-Laws was published in the NEWS LETTER, November 20, 1928. The purpose is to extend the balloting period so that distant members will not be disfranchised. The italic type and footnote references indicate amendments approved by the Convention, Feb. 14, 1919, for submission to letter ballot to be canvassed May 15, 1929.

[1] Change to "six months prior to the Annual Convention."

[2] Change to "120 days."

[3] Change to "60 days." [4] Change to "60 days."

cies in any office for the unexpired term shall be filled by the Board of Direction, except as provided above.

SEC. 5. The Board of Direction shall have general supervision of the affairs of the Institute and at the first meeting following its election, appoint a Secretary and from its own members a Finance Committee of three; it shall create such special committees as may be deemed desirable for the purpose of preparing recommended practice and standards concerning the proper use of cement for consideration by the Institute, and shall appoint a chairman for each committee. Four or more additional members on each special committee shall be appointed by the President, in consultation with the Chairman.

SEC. 6. It shall be the duty of the Finance Committee to prepare the annual budget and to pass on proposed expenditures before their submission to the Board of Direction. The accounts of the Secretary and Treasurer shall be audited annually.

SEC. 7. The Board of Direction shall appoint a Committee on Resolutions to be announced by the President on the first regular session of the annual convention.

SEC. 8. There shall be an Executive Committee of the Board of Direction, consisting of the President, the Secretary, the Treasurer and two of its members, appointed by the Board of Direction.

SEC. 9. The Executive Committee shall manage the affairs of the Institute during the interim between the meetings of the Board of Direction.

SEC. 10. The President shall perform the usual duties of the office. He shall preside at the Annual Convention, at the meetings of the Board of Direction and the Executive Committee, and shall be ex-officio member of all committees.

The Vice-Presidents in order of seniority shall discharge the duties of the President in his absence.

SEC. 11. The Secretary shall be the general business agent of the Institute, shall perform such duties and furnish such bond as may be determined by the Board of Direction.

SEC. 12. The Treasurer shall be the custodian of the funds of the Institute, shall disburse the same in the manner prescribed and shall furnish bond in such sum as the Board of Direction may determine.

SEC. 13. The Secretary shall receive such salary as may be fixed by the Board of Direction.

ARTICLE III

MEETINGS

SECTION 1. The Institute shall meet annually. The time and place shall be fixed by the Board of Direction and notice of this action shall be mailed to all members at least thirty days previous to the date of Convention.

Sec. 2. The Board of Direction shall meet during the Convention at which it is elected, effect organization and transact such business as may be necessary.

Sec. 3. The Board of Direction shall meet at least twice each year. The time and place to be fixed by the Executive Committee.

Sec. 4. A majority of the members shall constitute a quorum for meetings of the Board of Direction and of the Executive Committee.

Article IV

DUES

Section 1. The fiscal year shall commence July 1st.

Sec. 2. The annual dues shall be twelve dollars and fifty cents ($12.50) payable annually in advance from the first of the month following notification of the applicant of his election by the Board of Direction.

Sec. 3. Each member shall be entitled to receive one copy of one volume of the Proceedings for each membership year and additional volumes at a price fixed by the Board of Direction.

Sec. 4. A member whose dues remain unpaid for a period of three months shall forfeit the privilege of membership and shall be officially notified to this effect by the Secretary, and if these dues are not paid within thirty days thereafter his name shall be stricken from the list of members. Members may be reinstated upon payment of all indebtedness against them upon the books of the Institute.

Article V

STANDARDS

Section 1. Proposed new or revised Standard Specifications, Standard Practice, and Standard Definitions when approved by a majority voting in the committee in which they originate, shall be submitted, in the form adopted in the Standard Form of Standards, to the secretary of the Institute 60 days prior to the opening of the annual convention at which they are to be presented. The secretary of the Institute shall cause these proposed new standards or revised standards to be printed as Proposed Tentative Standards and mailed to the full membership of the Institute thirty days prior to the opening of the convention. As there amended and approved, they shall be published in the Annual Proceedings, next issued as Tentative Standards. At a subsequent annual convention, they may again be offered unamended, by their originating committees as proposed standards, and as there approved by a majority of those voting, they shall be submitted to letter ballot of the Institute membership, to be canvassed within ninety days thereafter. Such proposed standards shall be considered adopted unless at least 10 per cent of those voting shall vote in the negative.

Article VI

AMENDMENTS

Section 1. Amendments to these By-Laws, signed by at least fifteen members, must be presented in writing to the Board of Direction ninety days before the Annual Convention and shall be printed in the notice of the Annual Convention. These amendments may be discussed and amended at the Annual Convention and passed to letter ballot by a two-thirds vote of those present. Two-thirds of the votes cast by letter ballot shall be necessary for their adoption.

Summary of Proceedings of the 25th Annual Convention

The 25th annual convention of the American Concrete Institute was held at the Book-Cadillac Hotel, Detroit, February 12, 13 and 14.

Registration began Monday, February 11 because of the convention of the National Concrete Products Association with a large overlapping membership; thus a good deal of registration was taken care of before the Institute convention opened and avoided some of the usual confusion of the first few hours of registration on the opening day of the Institute meeting. Total registration was 685, of whom 69 were ladies.

About two hundred attended the opening get-together luncheon of the Institute in the Italian Garden of the hotel at noon, Tuesday, February 12.

Tuesday 2 p. m., February 12

President Edward D. Boyer, Chairman.

This session was built up largely through the efforts of the Institute Committee E-6, "Destructive Agents and Protective Treatments," the entire session being under the general title, "A Condition Survey of Concrete Structures." In introductory remarks the chairman of the committee, A. E. Lindau, explained the belief of his committee that its work for the time being should be directed chiefly to the discovery of causes for deterioration in view of the greater interest in preventive measures than in remedies.

The paper by H. C. Ash, Duluth, "Thomson Dam and Reservoir," had advance distribution in preprint. The author of the paper not being present its outstanding features were presented by F. R. McMillan, Secretary of Committee E-6. Mr. McMillan also presented the preprinted discussion of Mr. Ash's paper by M. B. Lagaard. Mr. Ash's paper was a review of the materials, methods and conditions involved in the construction of the Thomson dam and reservoir more than twenty years ago and Mr. Lagaard's discussion was the result of a recent examination of the structure after severe exposure since its construction.

L. W. Walter, Inspecting Engineer, Erie Railroad, Jersey City, N. J., presented a brief summary of his paper, "Thirty Years Field Experiences with Concrete." The paper was somewhat of an historical narrative of the reversal in field practice in the course of three decades in the development of making concrete.

Roderick B. Young, Testing Engineer, Hydro-Electric Power Commission of Ontario, Toronto, presented a summary of his preprinted paper, "Lessons from Concrete Structures in Service." The point of Mr. Young's paper was in the development of the fact that many concrete troubles can be prevented by some forethought and care in applying the newer methods and by proper supervision and greater attention to details. In the presentation of the paper Mr. Young largely confined himself to a brief presentation of the facts as disclosed in pictures of the various work covered by the investigation of structures in service.

The session as a whole developed considerable discussion. Included in the prepared and longer discussions were contributions by Dr. W. K. Hatt, P. J. Freeman, A. S. Brock, A. S. Douglass, Kristen Friis.

TUESDAY 8 P. M., FEBRUARY 12

A. R. Lord, Chairman.

The first paper of the evening was "Three and a Half Years' Experience of the Detroit Edison Co. in Concrete Control," by A. S. Douglass, Construction Engineer, and J. S. Nelles, Engineer, Detroit Edison Co. This paper was preprinted and had advance distribution. Mr. Douglass read portions of the paper and with the aid of stereopticon slides discussed methods used by the Edison company in quality control and results obtained.

L. G. Lenhardt, Engineer in Charge of Land Tunnels, Division of Engineering, Department of Water Supply, Detroit, presented in abstract his paper, "Concrete Lining of the Water Supply Tunnels of Detroit," which had been distributed in preprint. Mr. Lenhardt's paper told of the methods employed to insure a consistently high quality concrete in the water tunnels.

John M. Bischoff, Commissioner of Building and Safety Engineering, Detroit, presented his paper, "Supervision and Inspection of Concrete in Relation to Modern Building Construction." The paper told of the methods used by the City of Detroit in checking up on concrete work, both public and private.

Francis S. Onderdonk, Instructor, College of Architecture, University of Michigan and author of a recent work, "The Ferro Concrete Style," made an address under the title, "Is a Specific Ferro Concrete Style Evolving?" With a stereopticon Dr. Onderdonk showed a considerable number of very interesting pictures illustrative of architectural development with concrete in this country and abroad. He discussed their special characteristics and some of their possibilities indicating the trend as to the conception of concrete as an architectural medium. All the papers of the evening developed more than the usual amount of discussion.

WEDNESDAY, FEBRUARY 13

About two hundred convention people visited the Rouge plant of the Ford Motor Co. in the morning. They saw the new rod mill, glass plant and the assembly line and then in the busses which had awaited the party went to the Ford Airport to participate in a ceremony dedicating the concrete runways of the Airport. There was a brief address by William Stout of the Stout Air Services, Inc., the developer of the Ford tri-motor plane, and a response by President Boyer.

In the afternoon at 2 o'clock there were two concurrent sessions. Professor Duff A. Abrams, Vice-President, presided at the session devoted chiefly to Research.

In the absence of the authors, Raymond E. Davis, Professor of Civil Engineering and O. G. Troxell, Assistant Professor of Civil Engineering, University of California, their preprinted paper, "Volumetric Changes in Portland Cement Mortars and Concretes," was presented by title with a few remarks in connection with the importance of the work by Dr. W. K. Hatt, who also presented a discussion of the Davis-Troxell paper.

The preprinted paper, "Comparison of Methods of Determining Moisture in Sands," by William R. Johnson, Assistant Engineer, Research Laboratory, Portland Cement Association, Chicago, was briefly abstracted in the absence of the author by F. R. McMillan, Director of Research, Portland Cement Association. Written discussion was presented by George A. Smith and Cloyd M. Chapman.

"A Scientific Trial Method for Designing Concrete Mixtures," by R. E. Robb, Professor of Civil Engineering, Evansville College, Evansville, Indiana, which had advance distribution in preprint, was presented by title only in the absence of the author.

"Water Tables and Curves for Use in Designing and Estimating Concrete Mixtures," which had advance distribution in preprint, was presented by title in the absence of the author, Herbert J. Gilkey, Associate Professor of Civil Engineering, University of Colorado, Boulder, Colorado.

E. E. Bauer, Instructor in Civil Engineering, University of Illinois, Urbana, presented a brief abstract of his preprinted paper, "High Early Strength Concrete." It developed considerable impromptu discussion.

F. O. Anderegg, Senior Industrial Fellow, Atlas Portland Cement Co., Industrial Fellowship, Mellon Institute of Industrial Research, Pittsburgh, made a brief presentation of his paper, "The Mechanism of Corrosion of Portland Cement Concrete—With Special Reference to the Role of Crystal Pressure." This paper also had advance distribution in preprint.

H. F. Gonnerman, Manager Research Laboratory, and P. M. Woodworth, Assistant Engineer, Research Laboratory, Portland Cement Association, Chicago, were next with their preprinted paper, "Tests of Retempered Concrete"; presentation in abstract was made by Mr. Gonnerman.

"Concrete Studies at Bull Run Dam," a preprinted paper by T. C. Powers, Bureau of Waterworks, Department of Public Utilities, Portland, Oregon, was presented by title. Written discussion of the paper by Inge Lyse of the Research Laboratory, Portland Cement Association, Chicago, was presented by F. R. McMillan. Among the more extensive discussions of this paper was one by E. M. Brickett.

The final item on the program of this session was the annual report of Committee E-3, Research, H. F. Gonnerman, Chairman. The report is a summary of Research in progress and has appeared annually for several years in the *Proceedinjs* of the Institute.

CONCRETE PRODUCTS

The simultaneous session devoted to Concrete Products Manufacture with P. H. Bates as chairman, had large and interested attendance—larger than anticipated.

L. A. Falco, President, Decorative Stone Co., New Haven, Conn., presented a brief introductory paper, "The Development and Use of Commercial Cast Stone."

"The Physical Properties of Commercial Cast Stone," which had advance distribution in preprint, by John Tucker, Jr., and G. W. Walker of the United States Bureau of Standards, was presented in the authors' absence by Mr. Bates. It reported an investigation made by the Federal Specification Board, preparatory to drafting a specification for cast stone.

"Testing Concrete for Absorption" preprinted, was presented by the author, Fred Weigel of the Southern Cement Products Co., Knoxville, Tenn.

Along a similar line was the paper by Raymond Wilson, Associate Chemist, Research Laboratory, Portland Cement Association, Chicago, and also preprinted, presented in summary by the author.

All the first four contributions to the program laid a foundation for the fifth contribution, a report of Committee P-3, Concrete Stone, L. A. Falco, Chairman, C. G. Walker, Secretary, presenting proposed specifications for cast stone. After considerable discussion, particularly in relation to the matter of absorption, the specifications proposed by the committee were adopted as tentative specifications of the Institute (P3-A-29T).

Committee C-3, Treatment of Concrete Surfaces, presented "Proposed Specifications for Finish Coat Portland Cement Stucco," preprinted. This was offered by W. D. M. Allan, chairman of the subcommittee on Stucco (C3-C-29T). Specifications were adopted as tentative.

R. A. Foley, General Manager, Superior Products Co., Detroit, presented a brief paper, "The Effect of Curing Temperatures," relating some of the experiences of his company in a close examination of curing conditions and results in connection with products manufacture, particularly pipe.

Committee P-6, Concrete Products Plant Operation, Benjamin Wilk, Chairman, E. G. Lantz, Secretary, presented a report, "Effect of

Time of Mix on Non-Plastic Concrete," with particular relation to those mixtures commonly used in the manufacture of concrete masonry units.

E. G. Lantz, Secretary of Committee P-1, Standard Concrete Building Units, presented a report which discussed specifications for concrete brick, a matter in which the Institute committee has been endeavoring to act jointly with the committee of the American Society for Testing Materials. The same committee asked for adoption as a standard of the Tentative Specifications for Concrete Sewer Manhole and Catch Basin Block (P1-C-27T) as adopted tentatively in 1927. It was voted to submit these specifications to letter ballot of the Institute.* The committee also moved the adoption as Standard of Tentative Specifications for Concrete Building Block and Concrete Building Tile (P1-A-28T) which were adopted as tentative February 1928. These were referred to letter ballot.*

B. S. Pease, Chairman of Committee J-2, representing the Institute on the Joint Concrete Culvert Pipe Committee presented the second report of the committee embodying Specifications for Reinforced Concrete Culvert Pipe. They were adopted as tentative specifications of the Institute (J2-A-29T).

WEDNESDAY 8 P. M., FEBRUARY 13

Past President A. E. Lindau, Chairman.

"The Development of Specifications for Reinforced Concrete" by George J. Eyrick, Specification Writer, Smith, Hinchman and Grylls, Engineers and Architects, Detroit, which was preprinted and distributed in advance of the convention, was briefly summarized by Mr. Eyrick.

P. J. Freeman, Chief Engineer, Department of Public Works, County of Allegheny, Pittsburgh, presented, "Purchasing Centrally Mixed Concrete."

L. E. Williams, Engineer and Director, State Concrete Materials Co., Detroit, presented "Confusion of Specifications for Aggregate," indicating difficulties of the large producers of aggregate in meeting the many varied specifications of the present time.

Committee E-5, Aggregates, R. W. Crum, Chairman, F. H. Jackson, Secretary, presented a preprinted report, "Proposed Revised Purchase Specifications for Concrete Aggregates" (E5-A-29T), in brief summary by Mr. Jackson, who moved the adoption of the report including the Specifications as preprinted as tentative standard of the Institute, superseding the tentative specifications adopted February, 1926 (E5-A-26T).

In the absence of S. C. Hollister, Consulting Engineer, Philadelphia, who had written with brief preamble a suggestion for "Specifications for the Small Job," the contribution was presented by title.

A. R. Lord made an interesting presentation of the paper contributed by E. Freyssinet, Paris, France, "Long Span Arch Bridges in France,"

* To be canvassed May 15, 1929.

as translated by S. C. Hollister and put forward when the manuscript was not available for the previous evening's program. Presentation was accompanied by stereopticon slides from photographs of the work in progress. Discussion of the paper was followed by the adoption of a resolution presented by F. R. McMillan expressing the Institute's appreciation to the author of the paper for his excellent contribution. The chairman called on R. Claye, a fellow-countryman of the bridge builder, asking him to convey personally the appreciation of the Institute to Mr. Freyssinet. Mr. Claye made graceful acknowledgment.

"Continuity as a Factor in Reinforced Concrete Design," preprinted, was briefly presented by the author, Hardy Cross, Professor of Structural Engineering, University of Illinois, Urbana.

"Design of Reinforced Concrete Slabs" preprinted, was presented by title in the absence of the author, Joseph A. Wise, Professor of Structural Engineering, University of Minnesota, Minneapolis.

Similar disposition was made of the paper, "The Absolute Basis of Proportioning Concrete and Its Economy," in the absence of its author, Joseph A. Kitts, Kitts & Tuthill, Concrete Technologists, San Francisco.

THURSDAY 10 A. M., FEBRUARY 14

President E. D. Boyer, Chairman.

This session began with the consideration of the theme, "Standards of Performance of Concrete for Various Uses," a symposium of nine papers on characteristics of concrete needed for various types of structures and conditions of use and exposure.

The contribution, "Concrete in Sea Water," by H. E. Squire, Assistant Harbor Engineer, State Board of Harbor Commissioners, San Francisco, was presented in the author's absence by W. E. Hart.

In the absence of C. J. MacKenzie, Dean of Engineering, University of Saskatchewan, Saskatoon, his paper on "Concrete in Alkali Ground Water" was presented by R. B. Young.

F. H. Jackson, Senior Engineer of Tests, United States Bureau of Public Roads, Washington, D. C., presented his contribution on "Concrete in Pavements."

John G. Ahlers, Barney-Ahlers Construction Corp., presented the paper of which he was joint author with J. J. Lindon, Construction Superintendent, Barney-Ahlers Construction Corp., and Millard F. Bird, Vice-President, A. C. Horn Sales Corp., on "Concrete for Industrial Floors."

E. Grant Lantz of the Cement Products Bureau, Portland Cement Association, Chicago, presented his paper on "Concrete for Masonry Units" under the general theme.

The remainder of the symposium papers were deferred to the afternoon session.

BUSINESS

A business meeting occupied the first hour of the afternoon session February 14.

First was the consideration of amendments to the By-Laws (as announced in the November, 1928 NEWS LETTER), to extend the period of balloting as provided in Article 2, section 3, of the By-Laws, so that very distant members of the Institute would not be disfranchised by long mail time. The convention adopted the amendments and they were referred to letter ballot of Institute membership.

President Boyer announced the winners of prizes given to those who held the ten highest places on the Honor Roll in sponsoring new members in the year February 1, 1928, to February 1, 1929. H. F. Gonnerman was first with fifteen new members; John G. Ahlers, second, with fourteen; F. E. Richart, third, with twelve; H. B. Emerson, eleven; W. D. M. Allan, ten; A. W. Munsell, nine; John Tucker, Jr., eight; W. F. Way, eight; E. W. Bauman, six; J. H. Wasson, six.

Formal announcement was made to the convention by President Boyer of the death since the last previous convention of Richard L. Humphrey, first President of the Institute (1905–1915). Mr. Boyer proposed a committee to draft a suitable resolution and on unanimous approval of the convention named A. E. Lindau and R. J. Wig. President Boyer called Mr. Lindau, past-president, to the chair. Mr. Lindau called for the report of the tellers (H. S. Van Scoyoc and C. A. Wiepking) on the annual election. Mr. Van Scoyoc announced that the count of the ballots showed the election of Edward D. Boyer for President for one year to succeed himself; S. C. Hollister, Vice-President for one year to succeed himself; Harvey Whipple, Treasurer for one year term to succeed himself; Director, third district, J. C. Pearson, two years to succeed himself; Director, fourth district, P. H. Bates, two years to succeed himself; Director, fifth district, A. R. Lord, two years to succeed himself.

President Boyer, being recalled to the chair, presented the President's annual address. In the course of it he referred to the new publication plans of the Institute (as announced in the February 4 NEWS LETTER) and asked for general discussion of the desirability of Journal publication. The proposal of a special Board committee which had made the report had been approved in its entirety by the Board of Direction at a previous meeting. After thorough discussion the President called for a vote. It was unanimous in favor of the new publication plan.

Consideration of the theme papers was then resumed.

W. D. M. Allan presented a paper on the "Requirements in Concrete Work for Stucco" and in this connection referred to new specifications for finish coat portland cement stucco adopted at an earlier session of the convention.

The paper by S. C. Hollister, Consulting Engineer, Philadelphia, on "Concrete for Dwelling Houses" was presented by title in his absence. Discussion of Mr. Hollister's preprinted paper was presented by R. M. Thompson.

Nelson L. Doe, General Superintendent, Turner Construction Co., New York City, presented "Concrete for Reinforced Concrete Buildings." Contribution to the theme by N. D. Mitchell, Bureau of Standards, Washington, D. C., on "Concrete for Fire Resistance" was presented by title.

THE DINNER

At the Institute twenty-fifth annual dinner members and guests witnessed the presentation of the Leonard C. Wason medal for the most meritorious paper at the 1928 convention, to Franklin R. McMillan for his "Concrete Primer"; the Leonard C. Wason medal for Research to S. C. Hollister (in Mr. Hollister's absence) for "Advancing the art of bridge construction in the design, construction and test of concrete skew arch"; record of this work is contained in Mr. Hollister's paper, Vol. 24, Institute *Proceedings*.

The Henry C. Turner medal was awarded to W. K. Hatt for his "Pioneer work in reinforced concrete research; for a quarter century of devoted, outstanding and continuous service in developing the knowledge of concrete."

President Boyer expressed the gratitude of the American Concrete Institute to George H. Fenkell, Chairman, and the Detroit Convention Committee who had so ably contributed to the success of the meeting; to Mrs. Fenkell, Chairman of the Ladies committee, who had contributed so much to the enjoyment of visiting ladies at the time of the convention; to the Hotel management for its cooperation. The dinner program was brought to a close with an able address by William A. Frayer, Professor of European History, University of Michigan on "Engineering and the Social Sciences."

AWARDS

THE WASON MEDAL

AWARDED EACH YEAR TO THE AUTHOR OF THE MOST MERITORIOUS PAPER PRESENTED TO THE PREVIOUS ANNUAL CONVENTION

AWARDED, 1929, TO

FRANKLIN R. MCMILLAN, for paper, "Concrete Primer," presented to the 1928 Convention.

PREVIOUS WASON MEDAL AWARDS

1916 Convention Paper—A. B. MCDANIEL, "Influence of Temperature on the Strength of Concrete."

1917 Convention Paper—CHARLES R. GOW, "History and Present Status of the Concrete Pile Industry."

1918 Convention Paper—DUFF A. ABRAMS, "Effect of Time of Mixing on the Strength and Wear of Concrete."

1919 Convention Paper—W. A. SLATER, "Structural Laboratory Investigations in Reinforced Concrete Made by Concrete Ship Section, Emergency Fleet Corporation."

1920 Convention Paper—W. A. HULL, "Fire Tests of Concrete Columns."

1921 Convention Paper—H. M. WESTERGAARD, "Moments and Stresses in Slabs."

1922 Convention Paper—GEORGE E. BEGGS, "An Accurate Mechanical Solution of Statically Indeterminate Structures by Use of Paper Models and Special Gages."

1923 Convention Paper—J. J. EARLEY, "Building the Fountain of Time."

1924 Convention Paper—RICHARD L. HUMPHREY, for two papers, "Twenty Years of Concrete" and "The Promise of Future Development."

1925 Convention Paper—E. A. DOCKSTADER, for paper, "Reports of Tests Made to Determine Temperature in Reinforced-Concrete Chimney Shells."

1926 Convention Paper—A. BURTON COHEN, for paper, "Correlated Considerations in the Design and Construction of Concrete Bridges."

1927 Convention Paper—ARTHUR R. LORD, for paper, "Notes on Concrete—Wacker Drier, Chicago."

WASON RESEARCH MEDAL

AWARDED FOR "ANY ORIGINAL RESEARCH WORK OR DISCOVERY WHOSE SUBJECT MATTER FALLS WITHIN THE SCOPE OF THE INSTITUTE'S ACTIVITY, AND WHICH HAS BEEN WITHIN THE YEAR OR WILL AT THE NEXT CONVENTION BY INVITATION BE THE SUBJECT OF A PAPER BEFORE THE INSTITUTE IS ELIGIBLE FOR THE AWARD"

AWARDED, 1929, TO

S. C. HOLLISTER, for "Advancing the Art of Bridge Construction in the Design, Construction and Test of a Concrete Skew Arch," as reported to the Institute, 1928, in "The Design and Construction of a Skew Arch."

THE HENRY C. TURNER MEDAL

PRESENTED NOT OFTENER THAN ONCE EACH YEAR, FOR "NOTABLE ACHIEVEMENT IN OR SERVICE TO THE CONCRETE INDUSTRY"

AWARDED, 1928, TO

ARTHUR N. TALBOT, for "outstanding contributions to the knowledge of reinforced-concrete design and construction."

AWARDED, 1929, TO

WILLIAM K. HATT, for "pioneer work in reinforced concrete research; for a quarter century of devoted, outstanding and continuous service in developing the knowledge of concrete."

President's Address

By E. D. Boyer*

The honor done me by my election to the presidency of the American Concrete Institute has touched me deeply. Younger men than I might have been chosen for this office. But since I have been chosen, I feel that in making this choice, the membership has honored not only myself, but that my generation has also been honored for what it has given to and done for the cement industry.

Great changes have come in the cement industry during my lifetime. From an infant business, struggling against precedent, against competition, against all those indefinable prejudices that attach to anything which "has not been done before," I have seen cement and its utilization in concrete grow to a magnitude that is almost impossible to comprehend. It has been truly said that modern civilization, as we know it today, would be impossible without this mass employment of concrete, ever growing to greater and greater proportions.

In these past years, I hope that I may have contributed my part to the organization and systematization that is an essential part of an orderly growth. Certainly, I have sat in committee meetings for enough hours to make up one normal lifetime; and if my memory serves me, those who have worked with me in the selection, classification and unravelling of the apparently tangled skeins that have come before these committees for consideration would constitute an imposing roster of names of all who have written their names large in the industry and in the art.

Those days are joyous days to look back on. The meetings with my fellows; the arguments and debates pro and con; the joy of battles over unimportant points; the temporary flush of victories and the fleeting sting of good-natured defeats over some test as against another, and best of all the stimulation of companionship in some temple of good fellowship when the meeting was over—few of us can forget them and I, for one, never want to.

But under it all was the evolution of an art. Today that art is comparatively stable. Wide and successful usage along reasonably fixed lines has brought about a commercial art that may be pursued with profit to both buyer and seller. Cement as a product is tending towards a fixed article of commerce that may be had by all and is accepted by all. Design standards are coded so that safe structures are the rule, yet without unduly cramping superior skill or individual initiative. All this is true and all of this is beneficial in the main.

* President, American Concrete Institute.

But though I have spoken to you thus far as an old trouper and in a somewhat retrospective vein as of the past, I want also to speak to you as of the future as I see it and what must be the task if the concrete industry is to grow to its full and proper stature and breadth.

As I review the progress and growth just referred to, it appears to me that research and development in concrete have been confined to only a few agencies and that the progress of which we are all so proud has not sufficiently permeated the whole construction industry.

The researches of the Structural Materials Research Laboratory of the Portland Cement Association have been very thorough and of immense value in demonstrating in an orderly and understandable way, the laws of good concrete which before were only dimly seen and partly understood by makers of concrete immersed in pressing practical problems. At the Bureau of Standards, the Portland Cement Association Fellowship under the direction of Dr. Bates and Professor Bogue, a fundamental and scientific research is being conducted into the basic principles underlying the constitution of portland cement and its setting and hardening. This great work, because it is being conducted with almost unbelievable refinement and accuracy, has as yet hardly covered more than the preliminary stages. It is probable as this investigation progresses, the facts and principles developed will be applied to our everyday practical problems in concrete.

The many scientific and practical papers and discussions which occupy this and previous years' conventions of the American Concrete Institute, contain also a wealth of fact and data available to the construction industry.

But with all this research and progress in concrete, can we really say that it has reached more than a minute percentage of those responsible for work in concrete? Perhaps that may sound pessimistic in view of the tremendous amount of printed matter on concrete distributed by the Portland Cement Association and by this Institute. But surely if the results of this progress in research have reached afar, it has been heeded and followed by only a minority.

Some of the work in concrete which daily comes to my attention is of the smaller classes. While not spectacular or attention-getting, it is, nevertheless, that type of work which forms by far the greater part of the total use of concrete. The small builders who are responsible for this work though by no means unintelligent and unskilled in the use of concrete, rarely if ever have heard, and still more rarely heed, the importance of the principles developed by the tests and research of recent years. Are we in some way responsible for this condition because of a lack in bringing our knowledge to more direct relation and contact with field practices?

If this is true, the next great field for investigation and for research should be for those conditions where commercial considerations prevail and where laboratory conditions are almost wholly absent. At the present time the lowest bid and the lowest cost supersedes all else in the

mind of the buying public. It is a great tribute to the natural ability of concrete that it will endure and does endure in so many cases under such conditions.

I believe, therefore, that our Institute should encourage research in the commercial field and should foster the correlating of existing commercial practices with laboratory tests and conclusions, in order that a unified, dependable procedure may be evolved which will be so practical, so simple, and so sure of results that it will become the standard procedure of all engineers and architects, and even of the very small user of concrete.

As a step toward making more readily accessible the vast store of data and information contained in the papers delivered before this Institute since its inception, your Board of Direction for some time has been giving study to several plans for making this information available. If we can make it easier for our members, and for the engineering and construction world in general, to easily locate data on any subject treated in a paper presented to the Institute, we will have done our part toward the dissemination of the knowledge so laboriously collected and presented at the conventions. These are now available in the bound volumes of the *Proceedings*, which are properly indexed, but they are now so many in number and cover such a wide field of concrete practice and scientific investigation, that it is no easy task to locate all data on any particular subject sought for.

In view of these facts, your Board at its last meeting, recommended to the Institute the abandonment of the yearly publication of the *Proceedings* and the substitution of a monthly journal devoted to technical papers and discussions, abstracts of literature, news letters, and similar material, and at the business session yesterday their proposition was accepted by the membership and to my mind this will be a long step toward popularizing or making practical use of the scientific progress achieved by our members. Instead of presenting at intervals of twelve months a bound volume crowded with tests and data, the material will be spread out in ten installments—monthly, from September to June, thus insuring a more careful reading and a more complete assimilation of conclusions reached in our technical papers.

With this plan, any new facts, tests, or discoveries will reach our members much more quickly than if publication were deferred until the next convention.

So much for our plans for future papers—but how can the wealth of material contained in our past *Proceedings* be made more usable and accessible? One of the ways proposed for accomplishing this is to make a separate index or bibliography of subjects for the complete list of *Proceedings* from the year 1905 to date. Obviously this complete indexing and abstracting of so many volumes will be a long and laborious work which can be entrusted only to able hands. It is to be hoped that the details of this plan can be successfully worked out.

As men who are perhaps more interested in the practical aspect of concrete, we should not, in our efforts to secure a more universal use of

new knowledge in concrete, fail to encourage an appreciation of the artistic and beautiful. Most of our investigations are concerned with quality, and while this after all is the primary consideration, it must be remembered that the general public, whose funds make possible our concrete structures, are susceptible to the appeal of good appearance.

In the last few years we know that great strides have been made in our ability to make concrete a more presentable material from the standpoint of beauty of surface and modelling. The president of the New York Building Congress has said that buildings deteriorate in value more because of surface unsightliness than because of structural deterioration. This is a message which should be taken to heart, for it also means that buildings of concrete, which have a more presentable exterior and interior appearance than was possible a few years back, will find a greater extension of use and a greater value than can be had by any other means.

Of this we have many illustrations and not least is the great construction undertaken by the Department of Public Works, State of New York. Here not only enduring and satisfactory buildings are provided in concrete and concrete alone, but also at such a saving in cost that the appropriation made for the work is enabled to cover 18 per cent (approximately) more than it would in any competitive material. That is to say, these all-concrete buildings won out over all other competitive materials by a margin of 18 per cent and with, I believe, an advantage in appearance as well as fire-safeness and durability.

I hope, then, that the Institute will lend encouragement to every effort to make concrete a more dependable, a more uniform and, structurally, a more acceptable material than it is at the present and especially than it has been in the past. I am sure that this will be done and I hope that it may be done while I hold this present office as President of the Institute.

If I have seemed in this brief talk to have emphasized the needs of concrete and of the American Concrete Institute, it has been only because I know that we will see such progress in the coming years that our present knowledge will prove to have been only part of a great growth or evolution. Inasmuch as my life has seen and been part of this great industrial growth and evolution, I am thankful that an over-standardization and an undue contentment did not enter the industry, let us say, some fifteen years ago. Many sighed for it at that time and believed that with a rigid standardization put into force by one means or another, peace and prosperity would enter with this standardization and that all questions would be ended.

How greatly would we be the losers if that standardization had been brought about? In these past fifteen years, more has been learned about cement and about concrete than was learned in the preceding thirty years. That standardization on the uneconomical, narrow, and immature lines of fifteen years ago would be acceptable today, is unthinkable.

Yet today we stand on the verge of a new standardization in response to that same plea of bringing peace, prosperity and enduring happiness to

the industry through declaring, largely by force of propaganda, that all that is to be learned has been learned and that the book shall now be closed.

So long as the mind of man remains the brilliant, questing thing that it is this may never be done, for those who are slumberously content today will soon pass on and a new generation with new and better ambitions will displace them.

That is the history of the industry as I have seen it and that also is the history of the world.

And I know these new and better things will come, because I know they must come to meet our needs. In my daily task, I am constantly confronted with problems and ever new problems. Why did this happen? How shall this be done or that be accomplished? When may we hope to use cement and concrete for such and such a purpose so that the result we want may be had with concrete and without resorting to other materials that cannot but be less satisfactory by their very nature.

These, gentlemen, are problems of commerce, not of the laboratory. They are problems that are with every constructor; and as I have contemplated them, I have realized that the answer does not lie in an inflexible standardization, but in that forward march that goes beyond arbitrary standards and unlocks, one by one, the secrets of nature.

To the American Concrete Institute, I would therefore give the vision that is with me. By force of the experience of my life in the industry, this vision is of an industry better and broader, more careful in detail and more appreciative of the necessary correlation between the dicta of the laboratory and the unforeseen differences that attend field work than is the case today.

And this vision, too, is of a carrying on by both older and younger generations, a learning more and ever more and a building that knowledge into specialized and profitable work that shall be wholly and distinctively concrete. And if, in my lifetime, there may come before this body such summations of that knowledge as will create this broader and more useful knowledge, I shall feel well rewarded for whatever I personally may have been able to do in my day and in my time, which, since you have honored me as you have tonight, is far, very far from ended.

A Condition Survey of Concrete Structures

Symposium of Three Papers Arranged by Committee E-6

Progress Report Committee E-6

The concrete industry seems to be entering a phase of its development in which durability is becoming a question of major importance. Durability has rather generally been taken for granted as an inherent quality of concrete and we have concerned ourselves largely with factors affecting the strength of concrete and its behavior under external forces or loads.

We have discovered the principal factors governing variability in quality (especially strength) of concrete and now "job control" is the order of the day on important construction units. But we are gradually becoming aware of the fact that the problem of durability is more a question of resistance to external conditions other than loads, as for example the action of frost, the change in temperature, the forces of crystallization of salts, the action of water, etc.

We may be faced with the necessity of using different specifications of concrete for different types of exposure, but for the common forces of disintegration there is one rather simple requirement to give the necessary resistance—impermeability. If we can produce a concrete of relatively low porosity we can resist most of the disintegrative forces we have to deal with.

The Committee decided to continue the examination of structures under service conditions, in fact to broaden the scope of the investigation so as finally to obtain a fairly comprehensive survey of concrete structures, particularly those structures exposed to the more severe weather conditions.

A formidable list of destructive agents can readily be accumulated from our present literature on the subject. The Committee feels, however, that for the present the industry needs an answer to the question as to how our structures are standing up under normal rather than unusual conditions.

It is common knowledge that concrete may be attacked by diluted acids, sea water, alkali water, pure water (especially if allowed to percolate through the concrete). Concrete may be disintegrated by frost action, by faulty aggregate and cement and suffer deterioration by volume changes and mechanical abrasion.

But of all these agencies the most common are frost action, volume changes and the percolation of water through the concrete mass. These are the forces that attack nearly all of our so-called outdoor structures in which the concrete is exposed directly to the weather, and these are the

forces that the concrete must be able to resist if we are to have durable concrete structures.

By a comprehensive survey of concrete that has been in service under these conditions the Committee believes it can establish: first, to what extent our concrete structures are meeting service conditions; second, the reasons for failure to meet them; and third, the methods of modifying our practice to meet them.

The Committee realizes the magnitude of undertaking such a survey and analyzing the results. It may in fact prove impracticable, because of lack of personnel and funds, to carry on until the answer is found. But if the answer is found it will be by intensive and comprehensive study and field surveys supplemented by laboratory investigations.

For several years the Committee has reported in detail cases of concrete deterioration. At the same time examples have been given of structures of similar character and exposure to those that have been damaged, indicating that for some reason or reasons the fault is not with concrete as a material but with its composition and manner of manufacture in particular cases.

A considerable amount of information has already been placed before the Institute by the Committee and additional data are available in the literature on the subject. However, a great deal more field investigation should be made before we are justified in formulating anything approaching a complete report.

For the present the Committee is concerned more particularly with preventive measures rather than a "cure of the disease." There is abundant evidence that concrete properly made is durable. The task before the Committee is to indicate or develop a technique in the production of concrete structures that will insure permanence under given conditions.

Under the personal direction of the secretary of the Committee an extensive program of field investigation has been inaugurated and the results of some of this work made available to the Committee in the form of brief papers or monographs on particular projects. We have also been fortunate in securing the story of pioneer work in concrete by men who had charge of this work and an opportunity to observe from time to time the durability performance of their work.

A. E. LINDAU, *Chairman.*

Thomson Dam and Reservoir

By H. C. Ash*

The St. Louis River, after picking up numerous small tributaries in northern Minnesota, empties itself through a rocky gorge into the westerly tip end of Lake Superior, the "fond du lac" of early French explorers. The river drops 484 ft. in its last seven miles.

Jay Cooke, financier and promoter of the Northern Pacific Railway, was quick to see the possibility of power production, and in the early seventies acquired practically all the land on both sides of the river for a long distance through these dalles. As nothing was then known of electrical power production, plans were considered for the establishment of mills to use direct water power at favorable locations. Fortunately, no work was done at any of these sites except at Thomson near the head of the rapids where a 14-ft. wooden dam was erected and the power utilized for a time to operate a saw mill and slate-brick factory.

It was not until 1905 that actual construction work was begun for the purpose of utilizing the wasting power. The plan adopted at that time was completed in 1907. It created a reservoir above the then existing wooden dam $1 \times 1\frac{1}{2}$ miles in extent, the longer dimension at right angles to the river course.

The water of the river is taken from near the easterly end of this reservoir through controlling gates and an 80-ft. canal two miles long, to the lower head gates at the upper end of the pipe line. The pipe lines, a mile in length, the largest one 12 ft. in diameter, conduct the water to the power house and turbines with a working head of 372 ft., producing 69,000 h.p. An additional 80 ft. of the 484-ft. head is captured by a subsidiary power plant at the foot of the rapids.

The confining structures of the reservoir form a broad capital U with the upright lines of its U, which are the east and west ends of the reservoir, $1\frac{1}{2}$ miles apart. The river, entering the open side on the north, spreads out to meet low earth revetments on the east and west. The base of the U is a ridge of slate, from a few feet to 40 or 50 ft. above the general ground level and about 100 ft. across. This ridge had numerous breaks through it, the main river channel occupying the largest one. Figs. 1 and 2 show up and downstream views of the St. Louis River taken from a point about 1500 ft. below the dam site. These views show the general character of the river gorge.

In constructing the reservoir the various gaps were closed by concrete dams of different types, depending on conditions and necessity.

* Resident Engineer in Charge of Construction, Thomson Reservoir, 1905–7.

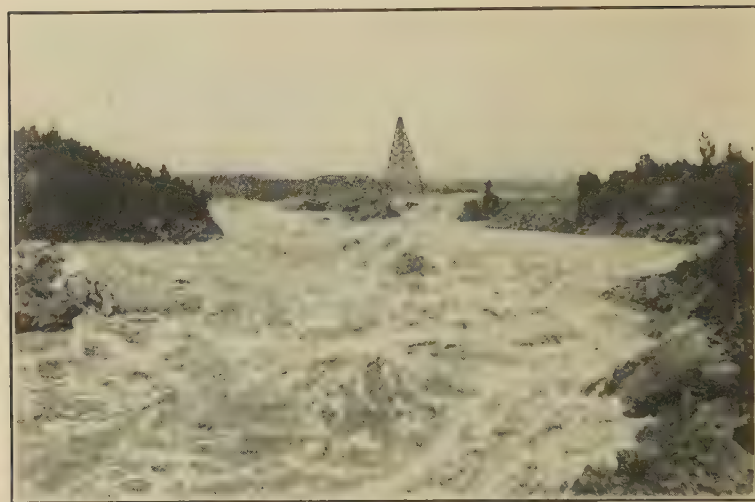

FIG. 1—ST. LOUIS RIVER LOOKING UPSTREAM TOWARDS THOMSON DAM SITES, SHOWING ONE OF THE CABLEWAY TOWERS.

FIG. 2—ST. LOUIS RIVER LOOKING DOWNSTREAM FROM NEAR THE THOMSON DAM SITE.

For purposes of identification and record the dams were numbered, beginning on the west with No. 1 and ending on the east with No. 12. Fractional numbers were used for the smallest structures, as, for instance, between No. 2 and 3 were $2\frac{1}{4}$, $2\frac{1}{2}$, and $2\frac{3}{4}$. Dams No. 1 and No. 12 are of earth. Dam No. 2 is an earth and rock fill with a 2-ft. concrete core

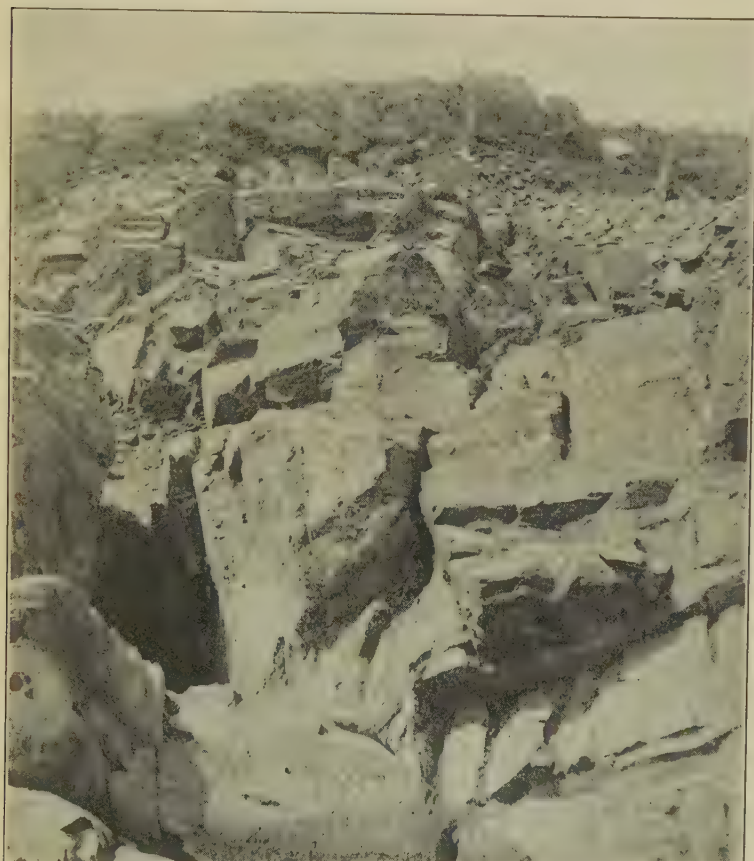

FIG. 3—A SECTION OF PREPARED FOUNDATION.

resting on hard pan. All other structures are of concrete, and dam No. 6 is of the arch type 54 ft. from base to top, 3 ft. wide at the top and 11 at the bottom, built on a 100-ft. radius. No reinforcing was used in this or any other of the concrete dams. All concrete structures were built on bed rock cleared of loose material and thoroughly scrubbed with brooms. A general view showing the character of the foundation as prepared for concrete is shown in Fig. 3.

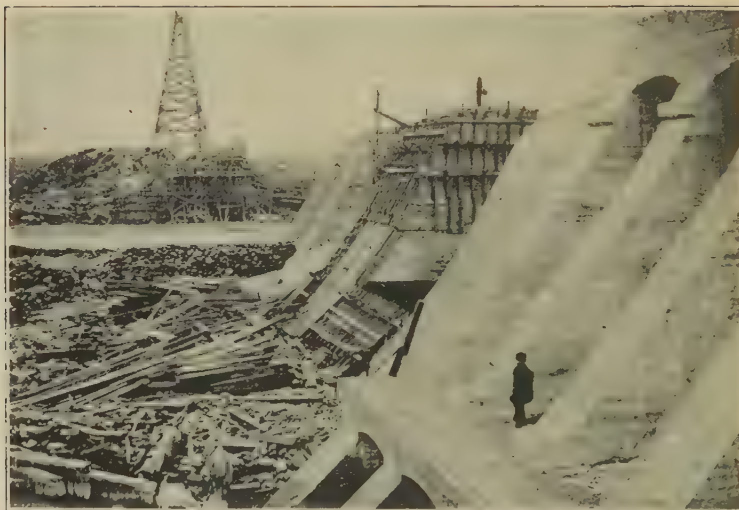

FIG. 4—CONSTRUCTION VIEWS SHOWING SECTIONS OF DAM NO. 3 AND THE END OF DAM NO. 4.

FIG. 5—CONSTRUCTION VIEW OF DAM NO. 3.

Natural conditions and the difficulties they involved had been studied and were pretty well understood before the beginning of construction. High June floods bringing down brush, dead timber, and runaway log booms, ice pressures from fields 2 to 3 ft. in thickness, anchor ice formed in the rapids above and low winter temperatures during the construction period were all expected. And none of them disappointed us by non-appearance.

In January, 1905, preparation for construction began with the building of camps, shops, warehouses, and railroad spurs. The old wooden dam which occupied part of the site of dam No. 3, the proposed concrete overfall, was blown up and the heavy ice 2½ ft. thick covering this location removed. This was a long and expensive job, but was felt to be necessary in order to construct a cofferdam before the spring thaw and thus leave the foundation of the proposed dam clear for operations.

A Flory cableway with high towers and 1000-ft. span was erected on the site to assist in clearing and later in handling the concrete skips. While this was in progress a very convenient and efficient mixing plant and a two-room cement testing laboratory were erected. The mixing plant was arranged so that a switch engine could shove the bottom-dump ballast cars filled with sand or gravel to the top of a stock-pile trestle. A continuous belt conveyor in a tunnel underneath the stock piles took either sand or gravel as desired to the elevator of the mixing plant where the aggregates were raised to storage bins equipped with a grid of steam pipes to be used in cold weather. From here the materials were dropped into measuring hoppers and from these to the mixer, which was a 1-yd. cubical. The cement which was brought in on a spur to the warehouse was taken by belt conveyor to the measuring hopper. The water for the mix was passed through a carefully calibrated barrel; a pull of a lever admitted the exact amount wanted, shown in pounds on an indicator.

None of us felt that our combined knowledge covered all there was to know about cement and concrete, and for that reason we may have been more careful than we otherwise would. Our laboratory consisted of a warmly built two-room shack. One room was used for a work room and contained two submerging tanks heated by a small hot water plant. Each tank was capable of holding 1600 briquets. One of the tanks was used for long time tests and small beams. In addition to the usual equipment of a field laboratory we had a 2000-lb. Riehle testing machine and a barrel of Ottawa sand. The second room was reserved for cement samples which were numbered and stored in case a repeat test should be wanted.

All cement shipments were tested both for time of setting and tensile strength at twenty-four hours, seven days, twenty-eight days, one year, and two years. Specimens were saved for a three-year or longer test. Comparative tests were made using Ottawa, lake, and various pit sands. Also tests were made of beams 6 by 6 by 48 in. using various sands, lake and pit gravels. Some interesting experiments were made on the effects of freezing and thawing during different stages of setting.

Records were kept of every test. We knew nothing then of field slump tests, the inundation system, or of the water-cement ratio. Some of our attempts at learning facts would seem crude now.

We built a watertight box holding exactly one cubic yard, filled it with dry sand or the various gravels, then poured in measured quantities of water in an attempt to ascertain voids. The large box was used instead of a smaller one in order to decrease the percentage of error in measurement.

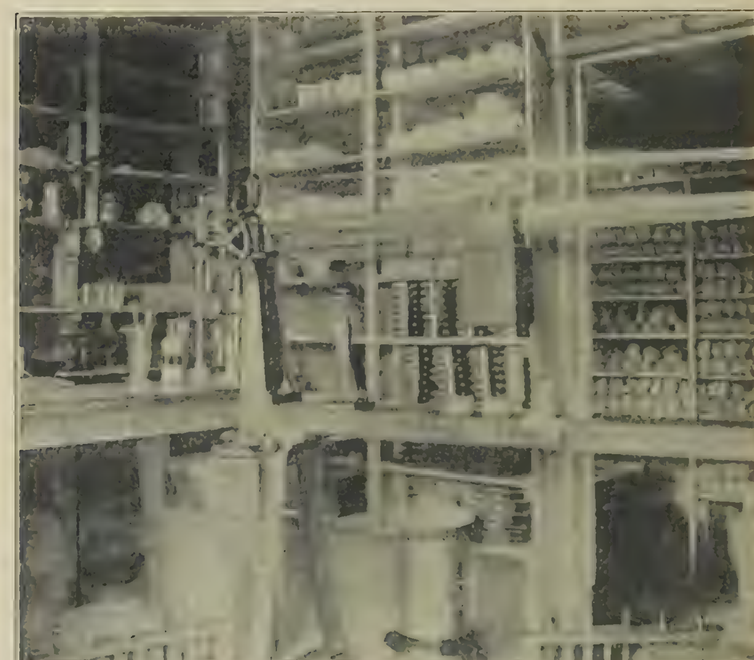

FIG. 6—INTERIOR OF CEMENT TESTING LABORATORY.

The various dams and retaining walls were built in 30-ft. long sections. The sections as well as the dams were numbered, and the records of the inspector at the mixer and the inspector at the form were so kept that it is possible today to tell exactly what entered into every foot of every structure. The records give date, dam and section number, elevation at the beginning and ending of each half day, car number of the cement used, number of mixes, proportions of cement, sand, gravel, and water, source of sand and gravel, kind of cement, men employed, air temperature, and the names of the inspectors. It was believed that the impossibility of shifting blame from one to another would induce more careful work.

Concreting was begun on May 6, 1905, in dam No. 4, section 4. Dam No. 4 joins the overfall dam No. 3 on the east. Section 2 was alternated with section 4. The proportions of materials, as determined for the inspectors, were 21.6 cu. ft. of lake gravel, 8.4 cu. ft. of lake sand, and 5 bags of cement; water was determined by trial to make the concrete of the consistency of thick mush which by the slump test would be perhaps four inches. As we had no way of telling the amount of contained water in the materials in the stock pile, it was very much cut-and-try to get the right consistency.

FIG. 7—THOMSON RESERVOIR LABORATORY. BREAKING 6×6×48 IN. BEAMS.
George A. Tuck, inspector in charge of testing.

The principal other source of aggregate, besides the lake, was known as the Swenson pit. This gravel contained an excess of sand. To 20 cu. ft. of this pit gravel, 4 cu. ft. of lake gravel was added, using 5 sacks of cement as with Lake Superior sand and gravel. These proportions were not altered throughout the entire work. The only variation allowed was in the water, which was occasionally changed several times each day. Dry concrete, requiring much tamping, and free water on the surface of placed concrete were both discouraged with emphasis. The cableway turned out to be the limiting speed factor in concreting. Five

FIG. 8—DAM NO. 6 OF THOMSON RESERVOIR.

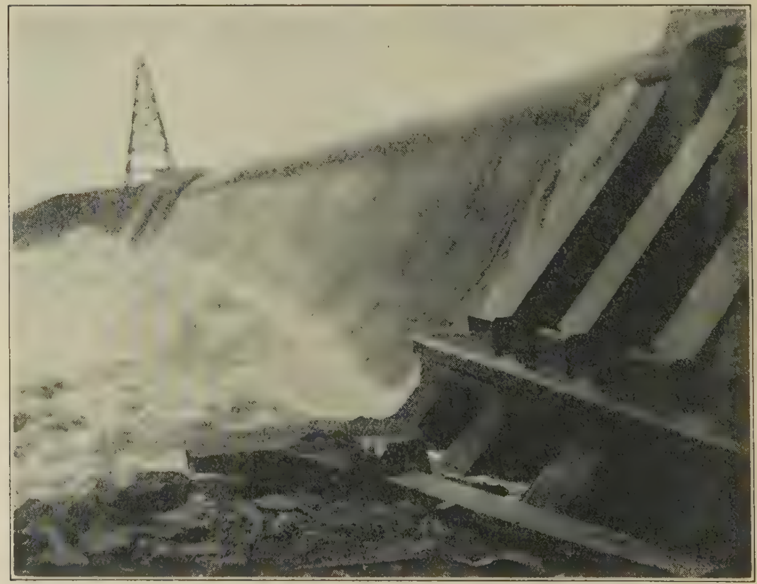

FIG. 9—SPILLWAY OF DAM NO. 3, THOMSON RESERVOIR.

minutes was about the average round trip time for the skip. This gave a minute to mix the concrete dry and three minutes after the water was admitted to the mixer.

The first six sections of dam No. 4, which were the first to be built, are the poorest looking today. The inspector in these forms unfortunately, as the engineer in charge considered it, had received his entire concrete training under a man who was an authority on dry concrete. On top of that he was Scotch and used to taking his porridge dry. The order that the concrete should be of the consistency of thick mush seemed to agree with his previous concrete training and his expert knowledge of porridge. It was not till a strong hint was given that he would have to change his ideas of mush or else look for some other occupation that a satisfactory understanding was reached. Those six sections show a number of areas where moisture percolates through, leaving a scaly incrustation. It is believed, however, that the concrete is sound and good for several decades longer. It is even quite probable that it is stronger than some of the better looking sections due to heavy ramming, but it has a disadvantage in appearance.

Concreting was continued through the summer of 1905 and the winter of 1905–1906, the last concrete for the reservoir enclosure being laid March 23, 1906. Work was rushed during the winter in order to be ready for spring high water, concreting continuing day and night. Through this period air temperatures sometimes ran low. From February 1 to February 6 the highest temperature reached was 1° F. and the lowest on February 6, −38°. All materials—sand, gravel, and water—were heated and the forms protected by canvas and heated with salamanders. The sluice gates were closed the night of April 2, ten days after completing concrete work. The water reached the crest of the overfall dam on April 4. Although this dam has had to withstand the tremendous "swoop" of thick ice pans passing over it during spring break-ups, no wear of any importance has yet taken place.

The arch dam No. 6 shows a vertical crack that appeared shortly after the reservoir was filled. No increase or apparent change has taken place since, and it is believed these structures are good for many years of useful service.

ADDITIONAL CONSTRUCTION DATA

(1) *Water-cement ratio*—Added water varied from about 2.7 to 4.5 gal. per sack. This apparently low water-cement ratio did not produce as dry a concrete as might be expected, for two reasons: first, the coarse aggregates were coming directly from the lake and consequently were quite wet; and second, the materials from the stock piles were taken always from the center and bottom of the piles where sun and wind had no drying effect.

(2) *Ramming*—Ramming was done with 20-lb. wooden rammers. The intention at all times was to produce a concrete that would not

require a large amount of ramming, one that could be readily worked into place and yet leave a reasonably firm surface. Sometimes this was accomplished and sometimes it was not.

(3) *Horizontal Joints*—Surfaces that had become set between runs were wet and covered with a 1:2 mortar before resuming concreting.

(4) *Vertical Joints*—Joints between sections were coated with asphaltic pitch.

(5) *Cement*—Cement used was Universal Portland throughout the entire job.

(6) *Pipe Lines*—The original pipe lines consisted of four circular California redwood stave pipes 84 in. in diameter, changing to riveted steel where the head required it. In 1925 there were constructed a 12-ft. circular steel pipe, a 11-ft. circular steel pipe and a 7-ft. wood-stave pipe.

(7) *Turbines*—The head is (gross) 372 ft. and the installed capacity of 69,000 h.p. includes: three 13,000-h.p. turbines, in the original installation, one 15,000-h.p. turbine, installed in 1914 and one 15,000-h.p. turbine installed in 1918.

Discussion—Thomson Dam and Reservoir

M. B. Lagaard.*—The paper by Mr. Ash is of unusual interest because he has given us, first hand, a description of an early construction in which particular attention was given to such details as the selection of materials, and the mixing, placing, and curing of the concrete. Mr. Lagaard.

In the late summer of 1928, the writer had the opportunity of examining this structure as a part of a nation-wide survey of concrete structures in service being conducted by the Research Laboratory of the Portland Cement Association. On one of the visits to the structure, the writer had the pleasure of being accompanied by Mr. Ash, which afforded an opportunity of obtaining the many intimate details of the construction operations, the importance of which are reflected in the present condition of the structure.

In observing the structure for the first time, the general impression gained is that certain portions are in excellent condition, while others are in a somewhat advanced stage of disintegration. Upon closer examination, however, it is developed that much apparent disintegration is in reality a localized incrustation and scale of no structural importance. Underneath this scaly deposit, as well as in all other portions of these structures except at the fill planes, the concrete is in excellent condition. In the exception noted, the concrete is spalled slightly in places, probably due to the freezing of water which has entered these seams. However, there is no evidence of laitance at these points.

In examining these structures, a number of photographs of typical conditions were taken.

Fig. 1 shows the east end of the spillway, which embraces what Mr. Ash has called Dam No. 3. Fig. 2 shows sections of Dam No. 4 adjoining Dam No. 3 immediately to the east or right. Fig. 3 is another section of Dam No. 4 still farther to the right. These views illustrate the range in conditions throughout the entire structure at the present time, Fig. 2 showing about the worst observable, while Fig. 3 is characteristic of the better part of the construction. The comments in Mr. Ash's paper regarding the placing of dry-tamped concrete are of particular interest in this connection, for Figs. 1 and 2 show portions of the first six sections which were placed by this method. Fig. 3 shows a portion of the structure where the concrete was placed in a consistency which Mr. Ash has described as being that of "thick mush."

The principal objection to placing the concrete by the dry-tamp method is the difficulty of obtaining a thorough bond between the succes-

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FIG. 1—SPILLWAY OF DAM NO. 3 AND WEST END OF DAM NO. 4.—THOMSON RESERVOIR.

Photographed August, 1928.—Note seepage lines in fill planes. See note under Fig. 2.

sive layers, even though a very high strength and impermeability may be secured in the mass itself. This is very clearly brought out in Fig. 2, where the bonding planes of successive layers are sharp and marked at the seepage lines. It will be observed that this comment applies to the successive layers of a few inches in thickness, as well as to the major fill planes marking the end of the day's operations. This photo shows that the greatest seepage has taken place through the major fill planes and it is immediately below these that the incrustation is thickest.

Contrasting the sections placed in the mushy consistency, shown in Fig. 3, with the dry tamp sections of Fig. 2 it will be seen that not only

FIG. 2—WEST END OF DAM NO. 4, THOMSON RESERVOIR.

Photographed August, 1928. Note heavy incrustation below fill planes and thin incrustation at seepage lines between layers of dry-tamp concrete.

are the major fill planes more watertight, as indicated by the extent of the deposit, but the section is entirely free from minor seepage lines.

Of particular interest also is Mr. Ash's comment to the effect that concrete in which free water accumulated on the surface and concrete requiring much tamping were both discouraged with emphasis. This care in the matter of consistency, together with the fact that unduly lean mixes were not attempted, in the opinion of the writer, accounts largely for the excellent results obtained.

In comparing Figs. 2 and 3, the question naturally arises as to the relative value of the concrete structurally in the two sections pictured. The superficial appearance in Fig. 2 would give the impression of a more or less serious disintegration, but as pointed out previously, when this surface incrustation was removed the concrete beneath was found to be in excellent condition. This is brought out rather clearly in Fig. 4

FIG. 3—PORTION OF DAM NO. 4, THOMSON RESERVOIR.

Photographed August, 192[illegible]. Concrete placed in consistency of thick mush. Note absence of minor fill planes and small seepage at major fill planes.

which shows a portion of the structure before and after the surface scale was removed. This was taken at a point in one of the first six sections to be constructed. It will be noted that after the scale was removed, the concrete underneath has the same appearance as that in the better parts of the structure, as shown by the other views. The form marks are clearly visible right up to the fill plane where the seepage had taken place.

Fig. 5 shows another view of one of these structures, which bears further evidence of the success of the methods which Mr. Ash put into effect. This shows the top of one of the small dams in the series in which

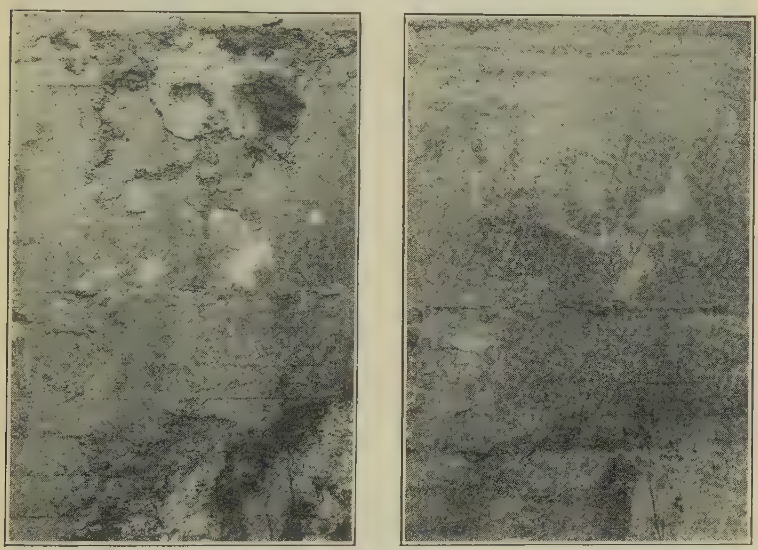

FIG. 4—BEFORE AND AFTER REMOVAL OF INCRUSTATION.

Left hand view shows portion of downstream side of dam with incrustation from water at seepage plane. Right hand view shows same spot with incrustation removed.

the surface of the concrete is finished by covering with heavy planking rammed in position and left while the concrete hardened. It is at once apparent in the photograph from the sharp corners and well defined form marks that the concrete is of excellent quality.

An interesting point in connection with this investigation was the discovery, near one of the small dams of this reservoir, of a section of one of the original test beams described in the paper. The peculiar reddish color of the aggregate identifies it as that of the original construction and the general appearance of the concrete resembled that in the structure. The surface was slightly worn, leaving the aggregate exposed. Through the courtesy of William H. Bachelder this sample was tested

FIG. 5—TOP VIEW OF ONE OF THE SMALL DAMS FOR THOMSON RESERVOIR.

Photographed August, [illegible]. Top finished by ramming heavy planking onto the surface. Note plank marks still visible and complete absence of scale or disintegration.

FIG. 6—ARCH DAM NO. 6 OF THOMSON RESERVOIR.
Photographed, August, 1928. Compare with Fig. 8 in paper by Mr. Ash.

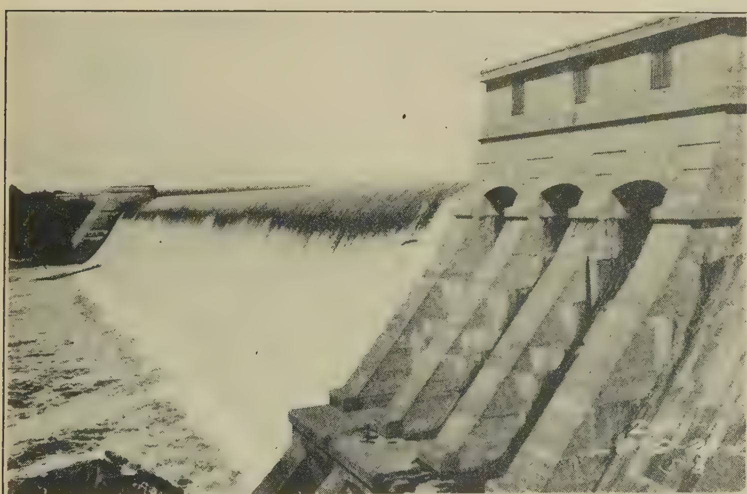

FIG. 7—SPILLWAY DAM NO. 3 OF THOMSON RESERVOIR.
Photographed August, 1928. Compare with Fig. 9 in paper by Mr. Ash.

for compression and for absorption in the laboratory of the Minnesota Highway Department. The test was made with a core $4\frac{1}{2}$ in. in diameter drilled from the beam. The ultimate strength in compression was 5850 lb. per sq. in., and the absorption 2.95 per cent. The failure showed shearing of the aggregate.

It is of interest to compare Figs. 7 and 8 taken last summer with Figs. 8 and 9 taken by Mr. Ash shortly after the structure was completed. Except for seepage at the fill planes, these recent views show no defects whatever. The excellent condition of the structure is evidenced by the fact that the corners and edges today are as sharp and clearly defined as they were at the time the original photographs were taken.

This record would not be complete without some comment in regard to the field laboratory which was established at the Thomson Dam. They not only tested their cement and their sand, but they made intelligent studies to determine the proportions to use, and further, the actual results being obtained were checked from time to time by the small test beams. The present highly satisfactory conditions of these structures is the reward for the intelligent efforts put forth in the careful study of the materials and control of the placing operations. This reward is undoubtedly a substantial one in the form of lower maintenance costs and longer life.

Thirty Years' Field Experience with Concrete

L. W. Walter[1]

During the last thirty years, beginning with 1899, the writer has been more or less closely connected with the study of materials used in making concrete and favored with the privilege of direct observation of field methods used in the making of scores of concrete structures and the opportunity of the follow-up system of inspection of these same structures.

During this period volumes have been written descriptive of methods employed in concrete construction and more volumes have been written descriptive of the work done in the field of testing, of research work and of the development of standard specifications involving an enormous amount of effort, indivudal and collective, so that today, from this accumulation of information available we should be able to make concrete with vastly more assurance of a high standard of quality than was characteristic of some of the concrete of the past.

This paper is presented with the hope that it may add somewhat to the common knowledge of today, through lessons learned in the past. It is hoped that it may be of interest as well to those of similar experience as to those whose memory and experience do not go back over such an extended period.

Early History—The year 1899 marks the year of a maximum production of natural cement in America. In the same year, 5½ million barrels of portland cement were produced.

Prior to that year a considerable amount of both natural and portland cement concrete had been placed. Concrete sidewalks and curbs were coming into quite general use. Street paving of brick and of granite and wooden blocks were being laid on concrete base. Government dams for slack water navigation had been built and others were under construction. Some foundations for stone structures being built were of concrete. Natural cement, puzzolan cement and portland cement were variously used. Tests of cement and of concrete were being made. Results of many tests had been published and were being read with a great deal of interest. All test results indicated that concrete of a dry consistency, if well compacted, requiring tamping to bring moisture to the surface, gave higher strength results than when more water was used in mixing. But many were skeptical at that time about the substitution of concrete for stone in new work and the engineers and architects adopting concrete for new usage were, by many others, considered venturesome. Those then using it, in their eagerness to get the very best results possible,

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adhered religiously to the then current specification requirements for tamping the concrete until moisture came to the surface.

In 1899 the speaker was present at a conference at which it was decided to build a concrete slab to be supported on rails spanning an 8-ft. opening between stone abutments in replacement of wooden track stringers. Justification for this lay in the fact that the rails would carry the traffic even though the concrete might not last very long. This is cited as an instance typical of the time.

It was not unusual at that time for the engineer to inquire of the stone mason as to what he thought about concrete and its suitability for new work being planned. Although a few reinforced concrete structures had been built, this branch of the industry was only in its infancy. This was 10 years before the first mile of rural highway concrete paving was laid in Wayne County, Michigan.

This same year, 1899, however, marks the beginning of the rapidly expanding use of concrete and of increased activities, particularly in the field of testing materials, to keep pace with the constantly increasing use of concrete. Test results were being published in increasing numbers and were being studied with widening interest and from that time up to the present there has been no general tendency to minimize the important need for good materials in concrete.

While some materials not in use 30 years ago have found favor in general use today, natural cement and puzzolan cement have gone almost entirely out of use. Each had merit for certain uses and could no doubt be made to serve useful purposes today. They have, however, passed out of general use, neglected but respected.

In the fall of 1902, construction of three new roundhouses was started in somewhat widely separated territory. The work on the three structures, which were of similar design, was done by different contractors under the direct supervision of different engineers. Three different brands of natural cement were used, one in each, and the sand and coarse aggregates obtained from different supply sources. Although it had been planned to have the concrete work finished in each of the three structures before freezing weather, the work was carried into the winter season and the concrete was damaged in part by freezing in each structure. Where the concrete was damaged it had been so disrupted by freezing as to necessitate the removal of all frozen concrete and its replacement with portland cement concrete. Two of these structures were located on sites where excavations to a general track level was necessary and one of the three was located where it was necessary to fill in a considerable part of the field area. In two of the structures, therefore, the foundation concrete in walls and pilaster footings was protected against freezing by early backfilling to the floor subgrade level. In the third, where filling was in later sequence, more of the concrete was exposed to frost action with the results that it was a total failure. During the winter and early spring, following the examination of the concrete in these structures, the speaker was privileged to extend his investigation to cover the experience of others who had used, in large quantities, each of the brands of cement

involved. Opinions expressed were not favorable to the use of natural cement in concrete where it would be subjected to freezing weather in its early stage of strength development.

Such of the concrete in the three structures as was not damaged by frost nor removed is still functioning after 25 years of service. After the experience in these three structures, the use of natural cement, by the railroad interested, was gradully curtailed.

Particularly important in this period of investigation has been the effort to work out the laws of sizing, grading and proportioning materials toward a more scientific design of concrete mixes, and today conclusions drawn previously from a study of voids, surface areas, and fineness moduli find common ground in the application of the water-cement ratio law.

The constant trend toward more uniformity and better quality of cements is seldom disputed. This improvement is marked by a slow process of evolution and the end is not yet. Creditable indeed have been the aims and efforts toward high quality but this is the age of research and we do not know what the future will produce.

Before discussing construction methods and their effect on the quality of concrete as observed from structures in service, it seems proper to state that while the conclusions drawn in this paper stress more the methods than the materials, it is not intended to infer that the use of any other than good material is justifiable.

Those who recall to their memory field methods and practices over the past 30 years will remember that at the beginning of the twentieth century hand mixing was generally resorted to, except in connection with the larger projects on which machine mixing was common.

Early Specifications—The following extracts from a specification of 1900 are of interest in showing the practice of hand mixing then largely in vogue. The specification is of particular interest in showing the care and detail given to the subject of tamping. This specification, which was prepared to apply in the construction of a concrete floor supported on concrete jack arches of a viaduct, had the approval of the five interested parties—three railroads, a state and a municipality.

> "One (1) measure of portland cement and two (2) like measures of clean, sharp sand (free from loam or other impurities), must be mixed upon a suitable platform or in a box, at least four (4) times or until the mixture has a uniform color; then shall be added five (5) like measures of stone broken to pass through a two and one-half (2½) inch ring, with the necessary quantity of water. The whole mass is to be thoroughly turned over with the shovels not less than four (4) times, or until the broken stone is completely incorporated. The resulting concrete when rammed, must give a slight surplus of mortar on the surface. If the size or the quantity of broken stone does not produce this result, then the quantity must be diminished. . . .
>
> "Each batch of concrete, after being mixed, must be put in place in horizontal layers, so as to give the requisite thickness when

it is thoroughly compacted; there must be at least one rammer to each batch. Any evidence of lack of compaction will be regarded as sufficient reason to require the removal and replacement of the base.

"The concrete must have such a consistency that, when rammed, it will not shake like jelly or be displaced laterally under the rammer"

This method of mixing and placing, and the specification for consistency were typical of the time. In other types of structures such as piers, abutments and retaining walls, it was customary to place concrete in layers of 6 to 9 in. and to use paving tampers to compact it. When the forms were removed, honeycombed surfaces were sometimes much in evidence and plastering naturally followed. Where neat cement was used in the plaster the color contrast was bad unless the entire surface exposed was plastered. This practice of plastering with neat cement proved objectionable on account of the unsightly appearance when the neat cement began to crack or come off. On some jobs mortar was plastered up against the form before the layer of moist concrete was put in place and tamped. Some preferred the use of metal sheets spaced about 1 in. from the form with mortar facing on the form side and concrete on the opposite, the two being placed simultaneously. The metal sheets were withdrawn and the concrete tamped into the mortar face.

The following is quoted from a description, furnished in 1928, by the engineer in charge at the work, of methods used in 1903, in building two concrete abutments of a railroad bridge over a highway.

"The proportions used were 1:2½:5 with gravel as coarse aggregate. The materials were all mixed by hand on platforms built above the forms and the concrete was shoveled from the mixing boards directly into the forms. Some of it required rehandling to shovel it to its final location in the structure. The concrete was placed in about 9-in. layers for the full length of the abutments and tamped with paving tampers. Special care was used, in placing, to work the coarse aggregate away from the forms for exposed surfaces and four to six men were employed in the work of spading and tamping. My recollection is that we placed about 25 cu. yd. of concrete per day. When the forms were removed, we wet the surfaces and applied lean mortar sparingly and rubbed it in and on with wood floats. This was not applied thick enough to be considered plastering and the board marks were only partially erased in finishing. The mixture was kept so dry that men walking in it, even after tamping, made only slight impressions in it."

This concrete examined in 1928 was in excellent condition as is shown in Fig. 1.

From the Drought to the Deluge—The method of placing concrete by thorough ramming of dry mixtures was tedious. It required special effort and close supervision. It is not surprising that those engaged in

this class of work welcomed the growing tendency to use more water. With the increasing confidence in concrete, the idea advanced next in order was that it might be all right to make dry concrete in the laboratory, but it was not practicable in the field. It was cheaper to handle it with a little more water in it and required less care and effort to get a good surface appearance. Some of our best concrete structures were built at

FIG. 1—RAILROAD ABUTMENT WITH STONE BRIDGE SEAT BUILT IN 1903. CONCRETE COMPACTED WITH PAVEMENT TAMPERS.

this time when water was being used in moderation, but the tendency gradually drifted toward wetter mixes.

Reinforced concrete was coming into use in building construction and the concrete used was made more fluid to be workable around the steel and into the narrower sections. Increasing quantities were being handled sufficient to warrant the use of concrete mixers, but the cost of conveying tons of concrete from the mixer to the job was a matter of

considerable concern. Then the engineer-contractor adopted the gravity system of conveying. Towers began to spring up and lines of chutes ran out to points distant from the mixer. By this time confidence in concrete as a material had advanced to where the average user felt that concrete was out of the experimental stage. Making good concrete was considered a simple matter. All that was necessary was to get enough cement and good materials and mix them well and most anybody could place the material in the forms. Concrete that 10 years before was so dry, after tamping, that one could walk across it without soiling patent leather shoes, was now so wet that knee boots were being discarded for hip boots for the men in the forms unless restraining influences were sufficiently effective to keep down the water content. Chutes were used at any convenient angle and the concrete was made to accommodate the chute. If dry batches came through and plugged the chute, the man at the mixer was blamed for the confusion and delay resulting. If the chute happened to have sufficient fall to carry stiff concrete, and concrete of a stiff consistency came to the forms, the workman in the forms complained and his demand for more water was oftentimes the last word on the job in water control.

The history of the first decade of this century is a history of transition from the drought to the deluge; the gradual merging from dry concrete into the exceedingly wet. The use of highly watered concrete was quite general over the next 10-yr. period, the concrete varying in degree of wetness with work done under different supervision. There were exceptions as to the extent of the abuse in some classes of work, such as in the making of concrete blocks, tile, small units made under factory methods by those specializing in this line of work, sidewalk construction and concrete road paving. Concrete highway paving which started with the second decade did not suffer so much from the use of too much water, but many highway structures did.

Engineering science of the first decade was directed largely toward the design of reinforced concrete structures, the application of concrete to new usage and the development of equipment and methods for lowering the production costs. The second decade marks the beginning of the application of corrective measures to get away from abusive methods in field practices. Those who were not active in the early application of corrective measures can scarcely realize the difficulties encountered in the effort to eradicate the abuses. The early adoption of corrective measures met opposition through seemingly every argument which human ingenuity could devise. Even today their enforcement is not easy. Eternal vigilance must characterize the future.

The following is a recital of observations made and conclusions drawn in relation to the adoption, by the Engineering Department of the Erie Railroad, of better methods in a determined effort to make better concrete. The enforcement of these measures, under duties delegated to the writer, required the close co-operation of a considerable number of men connected with the field forces.

A recital, at this time, of the conditions leading up to the adoption, 15 years ago, of measures considered revolutionary at that time, would hardly be warranted if assurance could not be given of a decided improvement in the quality of concrete resulting. Such assurance can be given.

In the earlier study of concrete troubles, where disintegration occurred in an occasional structure here and there, the case was studied as one exceptional and effort was made usually through the process of elimination to arrive at the cause or contributing causes of the trouble. These were matters of special concern. The scope of investigation was sometimes broad, covering conditions physical, chemical and electrical. Different investigators reached different conclusions. Some of the conclusions drawn did not seem to fit in later with other cases apparently similar and could not be reconciled with later developments.

Before the end of the first half of the second decade, it was apparent that disintegration in new structures was starting in much earlier and was progressing at a decidedly more rapid rate than was observed in older structures. The writer was then directly interested in quality features of concrete in typical railroad structures, but had been, for a number of years, engaged in the follow-up system of inspection of completed structures of general classification. The general investigation of field methods, from direct observation, over areas representative of all the districts in the United States east of the Mississippi River, and a careful inspection of outlying structures over an extended area furnished convincing evidence that increasing concrete troubles were general as to territory and that field practices, and not the quality of materials, were responsible for the pronounced change.

It was observed that disintegration first started in certain parts of the structures only. The concrete was cut out to examine it as to texture. Investigation was made of other types of structures elsewhere in large numbers. Out of these investigations dealing with the subject, the importance of which by this time gave serious concern, the conclusions drawn were: first, that concrete troubles could generally be traced to the use of too wet a mix; second, that disintegration was first setting in in parts of structures where light materials found final lodgment.

As remedial measures, the first and immediate aim was to cut down the water and adhere to what we termed a safe-working consistency to insure against segregation which results from a flowing action of concrete after it enters the forms.

It was obvious that the use of the chute would be immediately involved. Observations had been made of the flow of safe-working concrete in chutes of steel and wood of different dimensions and at different angles.

Under the existing contracts, it was held that the contractor could convey the concrete in any way he wished, provided the methods were not injurious to quality.

Field forces were instructed to notify the general office in all cases where chutes were being used. This notice was to be sent by wire, on or before the day concreting was started, if it was proposed to use chutes

at an angle of less than 35 deg. from the horizontal. Regulation of the consistency desired was effected at the mixer. Chutes slightly flatter than 35 deg. would not carry the concrete. At some structures the use of chutes was abandoned and other methods of conveying were installed.

Some of the concrete mixers would not discharge the concrete until the angle of the discharging chute was made to meet the new requirement. The discharge openings of most hoppers were too small to pass concrete of the new consistency. Specific attention of the contractor was called, at the time of awarding contracts, to the consistency clause of our specifications. Their field superintendents were told that if chutes were used in conveying concrete, the chute must accommodate the concrete of the consistency defined, and that the then quite common practice of making the concrete accommodate the chute at any convenient angle would not be tolerated. Where it was proposed to erect towers to elevate concrete for spouting, attention was called to the necessity for giving proper thought to plant layout and to need for sufficient height of towers to facilitate the flow of concrete of the consistency specified. Specific instances were cited where it had been necessary for contractors, even after previous warning, to increase the height of their towers to make use of them where located.

Legal advice dictated that in a specification clause covering consistency, the wording should be such as would admit of only one interpretation, if possible, and in the exigency of threatened suits, it should be such that the judge and the jury would place the same interpretation on it as would the engineer and the contractor. In the attempt to conform to this ruling the following clause was written in the specifications and remains in force today as applicable to plain mass concrete:

> "Concrete shall be of a viscous or sticky consistency. Concrete of this consistency will not be wet enough to flow to place within the forms, but will require shovelling or casting when being levelled to place. It shall be wet enough to give a good surface with careful spading and must be dry enough to require careful spading to secure a good surface."

The following clause was also added:

> "If chutes or spouts are used to convey concrete they shall not be placed at a less slope than thirty-five (35) deg. from the horizontal and hoods or baffle plates shall be used at the discharging ends to prevent separation of the aggregates."

The adoption of such policies and the employment of measures necessary to see that they were carried out was looked upon by some as strong-armed tactics because they were entirely out of line with practices at the time. The adoption of these measures arose out of conclusions carefully drawn. We had to be right. If we were wrong we had to get right. The more we investigated and the longer we studied, the stronger was our conviction that we were on the right track, but a campaign of education had to be carried on to convince those who did not take readily

to our conclusions. Substantiation of our views through other sources was welcomed as an aid toward their easier enforcement. As an indication of recommended practice at the time, the following is quoted from a manufacturer's catalog of 1914: "It is common practice to allow a grade for the spouting of 1 ft. of rise to 3 of run, which is approximately 18 deg. with horizontal. Fifteen degrees with the horizontal, or 1 to 4, is considered a minimum grade."

Another pamphlet of a later date (1917) contains a photograph of work in progress with an explanation indicating a fall of about 1 in 5 in the chute.

As an indication of the spirit of the times, the writer addressed the following letter to the Director of the Bureau of Standards, Washington, D. C., under date of April 1, 1914:

> "Careful investigation, covering a period of several months, of mass concrete constructed under the now quite prevalent practice of using a wet mixture and of spouting concrete to place leads the writer to conclude that, due to a departure from a safe-working consistency, much of the concrete placed during the last 3 or 4 years is of a quality quite inferior to that constructed prior to the advent of the chute as a means of conveying concrete and to the quite general practice of using a sloppy mixture.
>
> "I am of the opinion that investigation will confirm the claim that in much of the concrete placed under the present-day methods, a segregation of materials takes place after the concrete enters the forms as a result of which disintegration is setting in to an alarming extent in parts of many structures. It appears also to the writer that, in general, defects which are showing up in concrete placed during the last 3 or 4 years can be traced rather to methods than to materials and that the quite general abusive use of chutes and spouts contributes more to inferior quality of concrete than does any other single agent or factor.
>
> "Owing to the constantly increasing use of concrete in important structures it would appear that its quality and stability are of such general concern as to warrant a systematic investigation of the abusive methods now in vogue and the publication of reliable information as to their attendant evil results.
>
> "I beg therefore to suggest, for your consideration, that a comprehensive investigation be made by the Bureau of Standards of the evils resulting from abusive methods of placing concrete and to express a desire for such reliable data as may be furnished on this important feature as affecting the concrete industry."

It may be inferred from this letter that the writer is unalterably opposed to the use of chutes for conveying concrete. Such, however, is not the case. There is no quarrel with the chutes when properly used to convey concrete of safe-working consistency, but rather with the abusive use of the chutes at flat angles.

Fig. 2 is from a photograph taken of one of several retaining walls made of 1:2½:5 concrete placed in the years 1913, 1914 and 1915 at Jamestown, N. Y. The concrete here shown was handled through a mixing plant and tower mounted on a freight car and every yard of it was chuted to place. The chute was suitably baffled for discharging concrete into the upper sections. Flexible spouts of the elephant-trunk type were used to distribute the concrete from the chute to the lower levels in the wall. The minimum angle of 35 deg. with the horizontal was respected and a safe-working consistency of the concrete was maintained. An excellent quality of concrete was secured.

In searching the field for object lessons which would be drawn from older structures in which the consistency of the concrete and methods

FIG. 2—PORTION OF RETAINING WALL STAINED BUT WITHOUT DISINTEGRATION AFTER 14 YR. EXPOSURE. CONCRETE CHUTED BUT OPERATIONS UNDER GOOD CONTROL.

of placing seemed to play an important part, selection was made of a large number of small concrete arches built in the years 1907 and 1908 in the construction of a railroad.

In these structures it was found that portions of them were in exceptionally good condition, while other portions were beginning to show effects of disintegration. This disintegration was entirely limited in the structures to the tops and angles at the extreme ends of some of the bench walls which extended beyond the arch rings. The body of the bench walls and the entire arch rings were in exceptionally good condition.

The aggregates used were from different sources, much being of selected bar-run gravel washed, but used without screening and recombining. The matter of special interest in the study of these structures lay in the fact that the concrete placed in the bench walls was wetter

than that in the arches and the method of placing differed in the two. The bench walls were deposited between forms and as the tendency in

FIG. 3—END OF BENCH WALL EXTENDING BEYOND ARCH IN WHICH CONCRETE IS SOUND THROUGHOUT.

1907 and 1908 was to get away from the exceedingly dry mix, a somewhat wetter consistency was used in this portion of the wall than could be used in the arch rings which were placed without a back form and required

a very stiff consistency for the concrete to retain its position at the necessary slope. In working the concrete into the forms for the bench walls some excess water accumulated in the angles and tops and at the ends. To this condition was attributed the slight disintegration there, but not found anywhere in the arch rings.

In late years, most of these structures have been extended in length and now the original construction is almost entirely concealed. It has been difficult, therefore, to obtain representative pictures to show the points illustrated.

Fig. 3 shows the end of one of these culverts in which the end of the bench wall has been repaired. At the time the repair was made disintegration had not progressed very far but has since extended and shows the futility of repair work that does not take into account the possibility of future progressive disintegration. The arch ring does not show in Fig. 3, but it was similar to the arch ring in the other structures in that it is still free from such disintegration as shown in the bench walls.

Fig. 4 shows the side view of another culvert in which a portion of the arch ring is clearly indicated. The detail in the picture brings out the character of surface resulting from the dry placing without a back form, but with the use of a shovel to form the stiff concrete into place. The absence of disintegration will be noted.

The study of concrete troubles from structures in service with a view of designing and making concrete more durable requires the study and knowledge of conditions within and of destructive agencies without. Our present interest is in knowing how to make, and in seeing to it that there is made, concrete of such enduring quality and to such extent immune from the action of destructive agencies that a reasonable length of life of the structure can be assured.

It is not the intention of this paper to discuss all the various destructive agencies commonly encountered. It is desired, however, to give some thought here to the one agency often destructive to concrete in the earlier stage of strength development and sometimes even before the first set; namely, freezing.

Since the effect of early freezing is in most cases detectable soon after the concrete is placed, necessity for remedial measures is early indicated. Freezing of concrete surfaces subject to wear, such as in floors and paving, before the concrete sets is often damaging to the extent of necessitating complete removal and renewal. Freezing after the set, but before the concrete is sufficiently cured and of sufficient tensile strength to withstand the disruptive action of freezing, may be entirely destructive to the mass frozen, even though the damage may not be detected before the concrete thaws. Freezing of concrete in reinforced building construction must, at any cost, be prevented until sufficient strength is developed to insure its safety. Freezing of plain concrete before it sets is not necessarily damaging to the extent that its removal is warranted.

Fig. 5 shows a photograph of a specimen taken in 1928 from a back wall on a bridge abutment built in 1908 of 1:2½:5 concrete. The

speaker was present and saw the concrete placed. The thermometer registered 8 deg. above zero at the time and through lack of sufficient protection the concrete froze during the night and remained frozen for several days. In cutting out with hammer and chisel, the small specimen pho-

FIG. 4—END OF ARCH RING OF CONCRETE CULVERT EXPOSED 20 YEARS
Concrete cast in place without back forms and exposed surface was shovel finished.

tographed, a piece of hard trap rock was fractured without destroying the bond of the mortar attached. This is evidence that considerable strength did develop later in spite of the freezing before setting.

Safe-Working Consistency and Slump Control—After a mixture has been designed, suitable to meet the quality requirements for the concrete

in a structure, it is particularly necessary that great care be exercised in the placing of the concrete and in the control of the consistency to the end that segregation will not be induced through a flowing action of the concrete within the forms. Even when the consistency may be considered safe-working to the extent that segregation of materials other than water does not occur, the tendency of water to work toward the top may set up a wet condition at the top which will require corrective measures from time to time. If at any time excess water can be drawn off at the top, this should be done. Doing this, however, is not in itself sufficiently

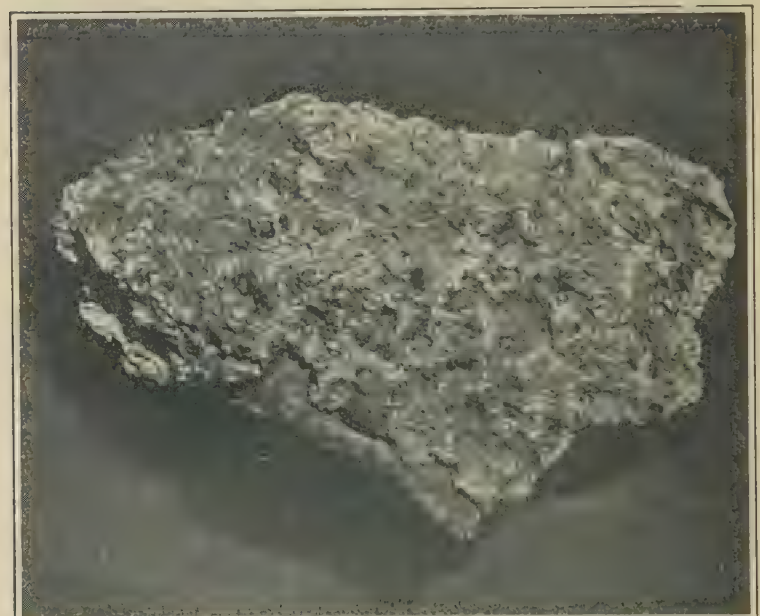

FIG. 5—IMPRINT OF ICE CRYSTALS IN CONCRETE FROZEN BEFORE SETTING.
Photo taken after 20-yr. exposure to outdoor conditions and locomotive gases.

corrective to insure good concrete in the top of the structure or at the top of a day's placing where there is no load superimposed upon it to compress it and squeeze excess water out of it while in its plastic state. To correct this overwatered condition in the top courses it will be necessary, from time to time, to place concrete of a lower water content and sufficiently dry that after absorbing the rising water, the ultimate water ratio in the concrete at the top will not be higher than that designed to meet the quality requirements. Necessity for a corrective measure, such as this, may be cited in numerous structures, such as in bridge abutments where 1:2:4 concrete in the bridge seats was placed monolithic with the

1:2½:5 concrete below the bridge seats. The effect of rising water, finding final lodgment in the bridge seats, is often more than sufficient to offset the intended value of a richer mixture placed in the bridge seats. The water-cement ratio law is basic and sound. Any unbalancing of the designed water ratio after the concrete enters the forms does not change the basic law but tends to defeat the results which could be expected from the proper application of it. In many cases where slump control is indicated it will readily be seen that the slump must be made to vary from a constant when dryer concrete for "corrective" needs is required. If a

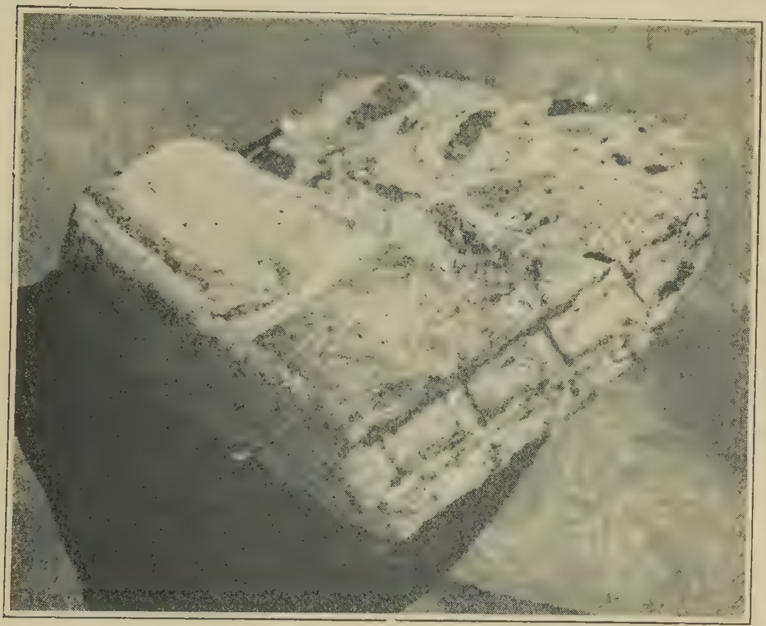

FIG. 6—SPECIMEN TAKEN FROM CONCRETE SHIP "ATLANTUS" 10 YR. AFTER LAUNCHING SHOWING IMPRINT OF SQUARE ROD $\frac{13}{16}$ IN AND OF ROUND ROD $\frac{5}{16}$ IN. FROM OUTER EXPOSED SURFACE. STEEL WAS NOT CORRODED.

maximum and minimum slump is named, the minimum should be low enough to allow for dryer concrete for "corrective" needs and the engineer in the field should be privileged, under the contract, to designate from time to time, a specific slump within the limits named.

Evidence that concrete can be made of quality appropriate to needs, under severe service requirements, and made under difficulties of placing, the equal of which was probably never elsewhere encountered and overcome, is presented in Fig. 6. This is a photograph of a specimen of concrete taken in 1928 from the concrete ship Atlantus, launched in 1918, it being the first concrete ship that was constructed under government

supervision. The imprint shown of the square bar was $\frac{13}{16}$ in. and that of the $\frac{3}{8}$-in. round bar was $\frac{5}{16}$ in. from the exposed surface. There was no evidence of corrosion of the steel in either bar.

As we look back at some of the work of the past and realize that much is known about concrete today in comparison with the general knowledge of it even a few years ago, we must admit that there is indeed much yet to be learned. With the lessons learned from experiences of the past, aided from time to time by knowledge gained through the ever increasing amount of research work that is being done, we can expect that this knowledge applied through the high intelligence in the engineering and contracting profession will result in concrete not only better than some of the concrete of the past, but sufficiently good to meet the requirements of the future.

Discussion—Thirty Years Experience with Concrete

W. K. Hatt—A large number of reinforced concrete beams were made in the year 1904 at Purdue University as part of the investigation conducted in connection with the St. Louis Exposition. The beams were 1:2:4 concrete made, as we used to make them in those days, of a live consistency tamped in the forms. These beams were broken in the laboratory and fractured until the steel was exposed. They were then discarded until recently I took one of them from the ground where it had been half imbedded for 24 years. Of course the steel that was exposed was very badly rusted. Eight inches from this rusted portion we chiseled away the concrete, exposing bars only 1 in. from the surface, which were as bright as the day they were placed. This indicates the possibility of protecting steel for twenty-four years, with one inch of well-made concrete of a live consistency. Prof. Hatt.

Lessons from Concrete Structures in Service

Roderick B. Young*

Anyone who critically examines a large number of concrete structures in service will be struck with the fact that most of the deterioration found therein can be classified into a few general types. If, as seems reasonable to assume, similar defects have their genesis in similar causes, the study of a few typical cases of each should show the basic cause for that class and indicate how they could be prevented.

This paper purposes to make such a study, but in order to keep to a reasonable length and to emphasize the points that the author wishes to bring out, the paper will only consider those defects that have their origin in the proportioning, mixing, placing and curing of the concrete. These probably make up over 75 per cent of all the defects in concrete; they are largely preventable, simply by care and forethought, and if they could be eliminated it would represent an important advance in the art of making concrete.

Fig. 1 is typical of a very common type of deterioration where the concrete disintegrates at the top of a "lift." In this case practically all of the structure reached by water is so affected. "Water-gain" during deposition seems to be the explanation of this phenomenon. Settling and consolidation of the concrete while still plastic causes water to rise to the surface, as anyone can observe by watching freshly-deposited concrete during a lull in the placing operations. The action occurs to some degree with all plastic mixes and is more pronounced as the consistency of the concrete becomes wetter. When the concrete is placed unduly wet, "water-gain" causes a decidedly "soupy" mixture to collect at the top of the concrete being deposited. Thus, we have the material at the bottom of a "lift" losing water to the material at the top, and according to the water-cement ratio law, the result is that the quality of the former is better and of the latter poorer than the average quality of the concrete in that location. The wetter concrete is porous and susceptible to frost action and thus the condition found in Fig. 1 comes about.

Fig. 2 is an unusual example of this type of trouble because it occurs in a retaining wall where the exposure is not particularly severe. The undercutting here extends to a depth of 16 inches. The concrete was made from crushed rock and screenings that contained a great deal of dust. It was put in very wet, as was evident from the fact that a 4-inch laitance seam existed under the overhang. Under these conditions, the

* Testing Engineer, Hydro-Electric Power Commission of Ontario.

top of the affected concrete was very weak and porous, and was attacked readily by frost action.

A common manifestation of "water-gain" is illustrated in Fig. 3,

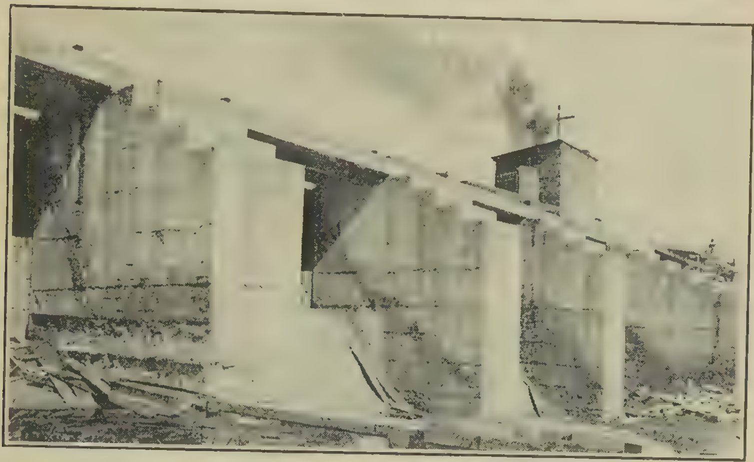

FIG. 1—WHERE CONCRETE IS PLACED TOO WET DETERIORATION STARTS AT AND PROGRESSES FASTEST TOWARD THE TOP OF THE DIFFERENT "LIFTS."

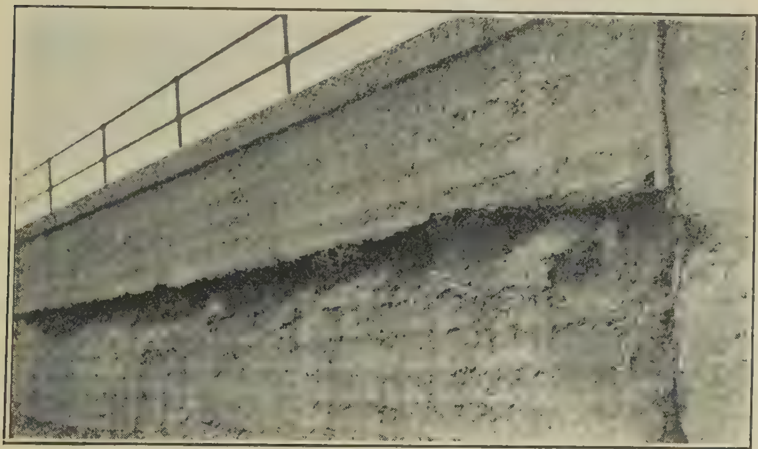

FIG. 2—THE CONCRETE AT THE TOP OF THE LOWER "LIFT" IN THIS WALL HAS DISINTEGRATED IN PLACES TO A DEPTH OF 16 IN.
Note how conspicuously the failing concrete is confined to the top two feet of the affected section.

which shows disintegration of the top and edges of a wall. Occasionally a similar deterioration occurs where the concrete mixtures are too lean in cement to withstand severe outdoor exposure, but the basic cause is

the same in both cases; namely, a porous concrete in a position where it absorbs water and is acted upon by frost or other destructive agent.

An excellent antidote to the effects of "water-gain" in a wall is to

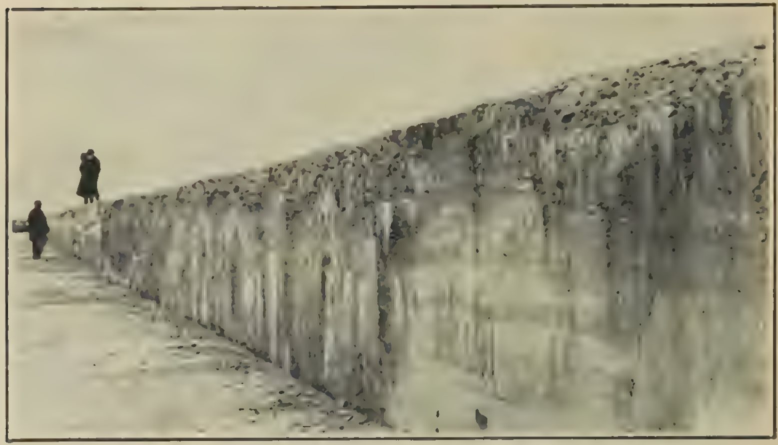

FIG. 3—WHEN CONCRETE IS PLACED UNDULY WET, DISINTEGRATION TAKES PLACE AT THE TOP SURFACE AND ALONG THE CORNERS.

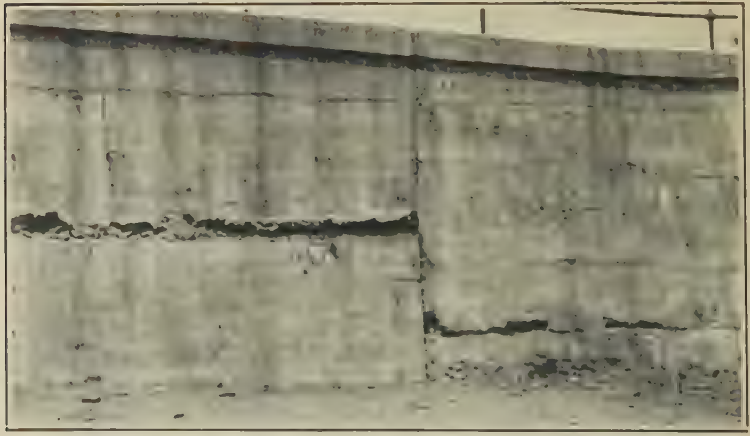

FIG. 4—MORTAR IS ALMOST TOTALLY LACKING IN THE CONCRETE IMMEDIATELY ABOVE THE LOWER LIFT OF THE LEFT-HAND SECTION OF THIS WALL.

overflow the forms and then strike off the poorer material that collects at the top; and another, sometimes applicable, is to increase the quantity of coarse aggregate in the last batches and to place them as dry as they can be handled.

Another condition that is frequently met with is deterioration immediately above a construction joint as in Fig. 4. At first thought, trouble at such locations would seem inconsistent with the theory just advanced for Figs. 1 to 3. However, as will be seen, the contributing causes are entirely different, the point in common being that the concrete is unduly porous at the point affected.

In Fig. 4, there is an almost entire absence of mortar in the concrete immediately above the joint. Apparently when the "lift" in question was begun, no attempt was made to provide the extra mortar needed when commencing operations by placing a layer on the surface of the hardened concrete below, and such an omission usually results in a poor joint.

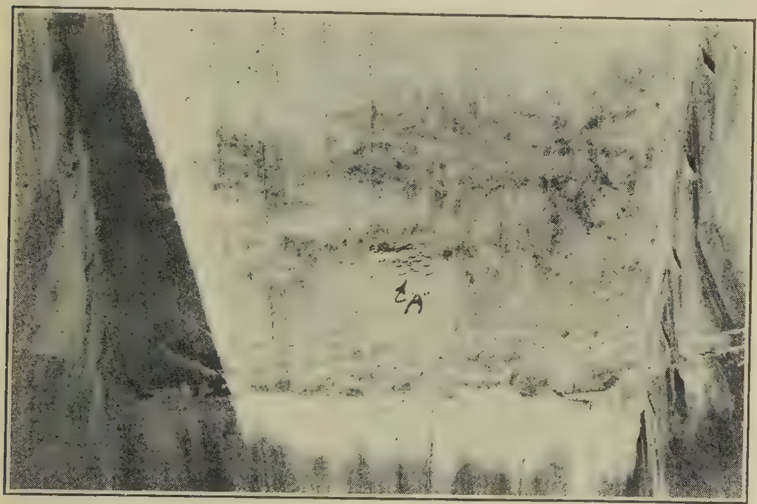

FIG. 5—DETERIORATION OF CONCRETE AT THE BOTTOM OF A "LIFT" BROUGHT ABOUT BY SEGREGATION.

The mottled appearance is caused by moisture which is coming through the concrete from behind.

Another factor might have entered if this were the first concrete deposited that day, for the first batch or two placed after a shut-down is partially robbed of its mortar by the amount that remains on the surfaces of the mixing and conveying equipment, and is sometimes so deficient in this respect as to make a proper joint an impossibility.

Another factor tending to cause greater porosity at the beginning of a "lift," is that at this time the forms are deepest and the concrete is less accessible to the workman and harder to puddle properly. The common practice of dumping concrete into the forms from the top also tends to cause segregation in the bottom sections.

Fig. 5 is an excellent example of deterioration brought about by the causes enumerated in the preceding paragraph. The concrete shown is

at the bottom of the deck slab of an Ambursen dam. At this point the slab is reinforced, is about 2 ft. thick, and lies on a 45 deg. slope. It was placed in 12-ft. lifts between forms. Under these conditions, it was practically impossible to puddle the concrete as it should be puddled, and a porous slab resulted.

FIG. 6—CONCRETE DISINTEGRATING AT THE BOTTOM OF A "LIFT" IN CORNERS WHERE THE SEGREGATED MORTAR COLLECTED.

The evidence of porosity and segregation in the concrete of Fig. 5 is very conclusive. The area marked "A" is a honey-combed spot that was covered originally by a thin mortar film, since broken away. The spaces between the particles of gravel are now partially filled with salt deposits showing that there has been a movement of water through the concrete which has leached out these salts. The mottled appearance of the concrete surrounding "A" is caused by porous areas which are satu-

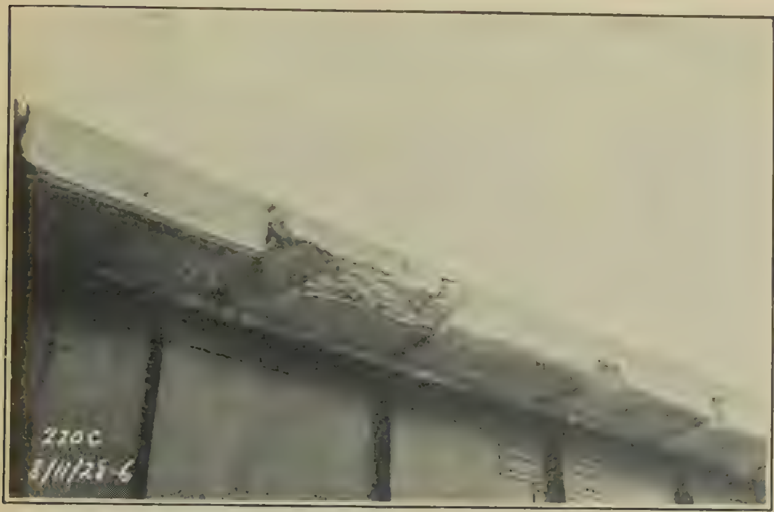

FIG. 7—DISINTEGRATION AT CORNERS AND ALONG EDGES DUE TO SEGREGATION IN FILLING FORMS.

FIG. 8—WHEN CONCRETE IS WORKED INTO A CORNER, THIS IS WHAT HAPPENS.

The top of the entire coping is disintegrating, probably due to being rather wet when placed.

rated with moisture, and the concrete is gradually deteriorating in these areas.

Fig. 6 is a third example of disintegration at the bottom of a lift caused entirely by segregation. The coping here was built with forms

FIG. 9—A SINGLE BATCH OF SEGREGATED CONCRETE HAS MADE THIS COLUMN VERY UNSIGHTLY, AND IS CAUSING DETERIORATION.

at both sides and top, the concrete was placed in the usual horizontal layers or lifts, and thus there were sharp corners between the forms and the coping into which it was difficult to work the latter. Cement paste and mortar from the concrete got into these corners and was later disintegrated by weathering.

Segregation of the cement and mortar from the concrete is a very frequent cause of deterioration because the separated mortar is usually porous and of low strength, and the concrete left behind is undersanded

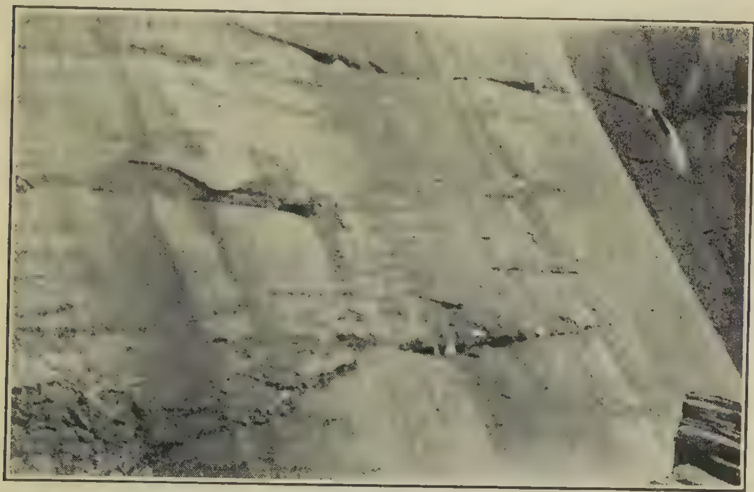

FIG. 10—LAITANCE POCKETS AND STONE STREAKS CAUSED BY ALLOWING THE FORMS TO FILL FROM ONE POINT.

FIG. 11—THE CONCRETE HERE WAS DEPOSITED AT ONE POINT AND ALLOWED TO FIND ITS WAY TO THE OTHER PARTS OF THE FORMS, SEGREGATING IN THE PROCESS AND FORMING STONE POCKETS AND STREAKS.

and honeycombed and therefore also porous. Trouble from these sources ordinarily develops in corners as in Fig. 6, along edges and joints as in Fig. 7 and Fig. 8, in fact at any location where the forms cannot be filled readily.

Segregation not only occurs at corners and edges, but may happen anywhere if the concrete is not properly handled and placed. In Fig. 9 is a large pre-cast column in which deterioration has started at the white area in the center due apparently to a segregated batch that got dumped in this location and which was not properly puddled. Extended observation of the concrete in this area has shown that it absorbs moisture readily, and Fig. 9 shows it after a prolonged spell of dry weather when the absorbed moisture had had a chance to dry out and bring the salts dissolved by it to the surface.

Another type of trouble from segregation is shown in Fig. 10. The forms were filled from one central point, and the concrete allowed to flow

FIG. 12—THE RESULT OF FINISHING THE SURFACE OF EXPOSED CONCRETE WITH A STEEL TROWEL.

In all other respects this concrete was excellent.

at will. This method of handling sometimes causes laitance pockets as in Fig. 10, or aggregate pockets as in Fig. 11, or both.

It will be noticed that practically all of the cases cited so far might properly be ascribed to one set of causes—porosity of the concrete brought about by segregation. Porosity and segregation are most important factors in determining the behavior of concrete in service. They can usually be traced to one of the following causes: improperly designed mixtures, an excess of mixing water, undermixing or careless handling and placing. The remedy is obvious and with our present knowledge there is no reason why any of these conditions should be allowed to exist on any job.

Scaling of surfaces is another type of defect frequently encountered. Fig. 12 is a typical example—in this case caused from using a steel trowel

to finish concrete exposed outdoors. The surface of the concrete has either been overworked or worked at the wrong time, or both. Steel troweled surfaces are not a success outdoors except with comparatively rich mixtures such as are used for side-walk tops. Except in the latter class of work, it is only occasionally that one will be found in good condition, and these instances are so few and far between that it is safe to assume that a steel-troweled surface will not prove durable where exposed to the weather. On the other hand, floated surfaces seldom scale and are the most satisfactory for outdoor concrete.

Fig. 13 is another type of scaling caused by overmanipulation. Here is a job where an overzealous engineer required the concrete next the forms to be puddled to excess, and built up at the outer surface a

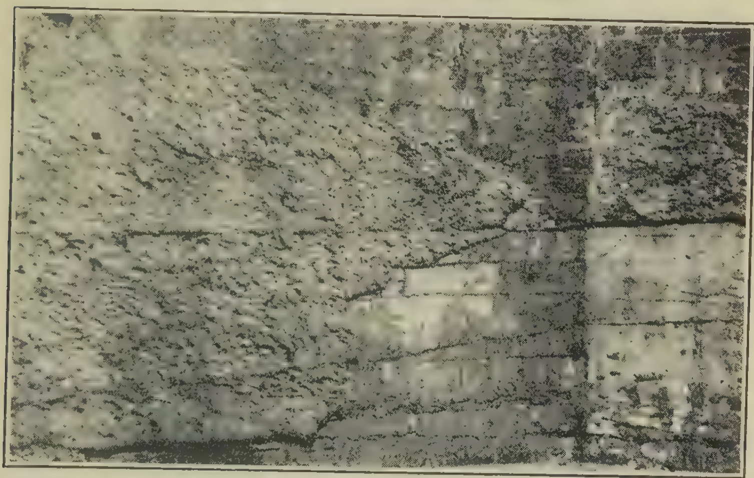

FIG. 13.—THE CONCRETE WAS OVERWORKED AND A ¾-IN. LAYER OF MORTAR FORMED AT THE SURFACE, WHICH LATER SCALED OFF. The concrete in this structure was generally of poor quality.

¾-in. mortar layer which later scaled off. Overmanipulation is as much to be avoided as no manipulation. Concrete is puddled to consolidate it and to obtain good surfaces, and the material should not be worked any more than is necessary to accomplish these purposes. Any considerable puddling of the concrete in the forms tends to cause segregation with its resulting evils. Nor is much puddling necessary, for if the concrete is well designed, fully mixed and properly handled, only a limited amount of working will be required to consolidate it and get a proper finish.

Fig. 14 is an example of scaling that illustrates another condition constantly to be guarded against on concrete work. The structure in question is a breakwater with the usual sloping face toward open water. The sloping face was built against a form; the sand used was fine, the

concrete of a fairly wet consistency, and mortar and laitance, which under these conditions collected under the upper form, have since scaled off. A similar condition existed in Fig. 6 toward the top of the coping. It is a matter of greatest difficulty to prevent the collection of laitance and muck under forms with a batter or having projecting edges or offsets but it must be done if trouble is to be avoided for such accumulations are sure to scale or disintegrate later.

The final step in the manufacture of concrete is the curing, and except in highway construction, it is probably the one most neglected. Much concrete is damaged by the methods followed in curing, either from allowing the concrete to dry out prematurely or from not keeping it at a

FIG. 14—WHEN MUCK AND LAITANCE COLLECT AGAINST A SLOPING FORM SCALING OF THIS TYPE USUALLY OCCURS.

sufficiently high temperature during its early life. Most engineers recognize that it is necessary for concrete to have a supply of moisture during hardening, and it is also generally realized that in cold weather concrete must be deposited warm and kept warm for a time. What does not seem to be fully appreciated is that, in the fall when cool weather prevails, concrete can be seriously damaged without being actually frozen.

The concrete shown in Fig. 15 was damaged in this way. A number of small piers were placed during mild weather in October, but a few days later cooler weather set in and from then until the following spring the temperature of the concrete was seldom above freezing. Under these circumstances, if it gained strength at all, it did so very slowly. In the spring, water was in contact with the base of these piers; they became saturated with moisture, and after a few freezing nights in this condition several failed as shown in Fig. 15, and others, though apparently not

damaged, have since so deteriorated that all will have to be replaced shortly. This case is not an isolated one for the author investigates an average of two or three each spring and in some of these, several thousands of dollars have had to be spent to replace the damaged concrete.

In each of the fifteen cases cited, an explanation has been given of how the defects came about; but in each case the fundamental cause was that the concrete, for the reasons given, was unduly porous and permeable to moisture or other destructive agency. Practically everything tending

FIG. 15—A PIER BUILT IN THE FALL THAT WAS NOT ACTUALLY FROZEN BUT WAS NOT PROPERLY CURED AND WAS DAMAGED BY FROST WHEN SATURATED WITH MOISTURE THE FOLLOWING SPRING.

to destroy concrete attacks it through its pores and this is true regardless of whether the ultimate destruction of the concrete is brought about by frost, sulphate or sea water, corrosive water or chemicals, or the disruptive effect of reinforcement or unsound aggregate. Even volume changes are, in part at least, due to differences in the moisture content of the concrete. It is evident, therefore, that in most cases the phrase "durable concrete" is synonymous for impermeable concrete, and this should be the goal of every engineer who has to build for outdoor service.

If the facts brought out in this paper are not new or startling, it should be remembered that most poor concrete is poor, because of the neglect of those elementary and well-established rules for its manufacture

that should be known to everybody, and not because of any obscure ailments to which it, as a material, is subject. It is not to be inferred by this that all deterioration of concrete is as simple as the cases cited or that concrete does not have obscure ailments, but after having examined many scorès of concrete structures of a variety of ages and conditions, the author has been unable to reach any other conclusion than that at least 75 per cent of the concrete in trouble today is so because its makers have permitted a few entirely preventable conditions to occur during its construction.

Discussion—Lessons from Concrete Structures

P. J. Freeman—It is very interesting to pass over the last five or six years and see how we have gone from just a very brief discussion of the troubles of concrete to spend one whole afternoon in discussing them. I heartily agree with the speakers. I think we can say that even more than seventy-five per cent of our troubles are sins of omission and commission which never should have happened. However, nothing has been said this afternoon about the material. Of course, in general, we do not have the facts about the material, but I simply want to bring up just one incident where we do have all the facts, in this instance about a certain limestone. Sections of a road were built with this limestone, as were also parts of a retaining wall in juxtaposition to other sections of the highway and of the retaining wall which utilized a different limestone. All other conditions were identical—the same contractor, the same cement, the same methods and the same amount of water. The concrete using the first limestone disintegrated badly, while that using the second remained in perfect condition. Mr. Freeman.

An examination of the quarry from which the first limestone had been taken showed that it was about thirty feet high and extended for about three miles along the railroad. Portions of it which had been exposed to the weather for about nine years indicated a serious sloughing off. When the quarry was opened for this work, tests were made showing the abrasion loss was less than 3 per cent, whereas most highway departments allow 5 or 6 per cent. The crushing strength was from 18,000 to 25,000 lb. per sq. in. or more. The toughness was far above specifications, and every requirement known at that time was met by the stone. However, when the engineer of test of that state arrived he noticed the outcrop of the quarry and said that he believed the stone was unsound. He tried it out with the sodium sulphate soundness test and found that it failed on about the second test. Therefore he stopped the work and got stone from a new source. I call your attention to the fact that the stone passed all other tests satisfactorily.

The stone itself looked somewhat like fire clay. It was an argillacious limestone, which in general would indicate a good stone. In order to secure a little more information on the subject, we interested the geologist of the U. S. Geological Survey, who found that sections of the material contained a clayey material known as bedellite, which can quite readily be picked out by a petrographer. Insofar as the limited studies we have made are concerned, bedellite and unsoundness go together.

I simply want to call your attention to this one example, not to cause any undue alarm about limestone, but to urge that before using a local quarry its outcrop should be studied.

Mr. Lord. A. R. LORD—I would like to ask Mr. Young if, in connection with the use of the steel trowel, he includes the case where mortar surface rollers are used on sidewalks?

Mr. Young. R. B. YOUNG—No, I think that where the mortars are fairly rich we have none of that trouble. It is when we use the steel trowel on ordinary concrete that the surface almost always scales.

Mr. Brock. A. S. BROCK—Several suggestions have been made with regard to controlling the water-cement ratio of the mass of concrete after it is placed. One was to drain off the surplus water. I have seen structures such as flat decks, like you get in a factory or a loading platform, where it is impossible or impracticable to drain off this surplus water. In every structure of that type it has been necessary, because of the intricate amount of reinforcing steel, to place the concrete with a fair slump. Concrete such as is being used on highway work, with about 2 in. slump was not suitable. It was necessary to have a 4 to 6 in. slump. In every case that I have observed, there has been an accumulation of water and possibly water and cement from ⅛ to ¼ in. in thickness.

Having found no practicable way of eliminating this water, I have about come to the conclusion that if that structure or that particular part of the structure is to be used for a wearing surface, a high-grade surface can only be obtained by applying a finish layer whose quality is carefully controlled. On the other hand, if it is necessary, as it often is, to apply a finish immediately, I have found no way yet to produce a high grade surface. That applies not only to resistance to wear but resistance to the weather. I would like a few suggestions along that line.

Mr. Douglass. A. S. DOUGLASS—In our work we have that particular problem probably much more extensively imposed on us than in the usual concrete work, because it is located in electrical substations and in power houses where we have not only a great amount of reinforcing but a large number of conduits. For work under these conditions we have found the use of air hammers for vibrators to be of great assistance. These air hammers fitted with flat ends are placed on the reinforcing, and the effect that we get by letting the flatter portion of the mixture flow between and below the conduit and steel is very satisfying. It permits the use of a drier concrete in slab work than we are otherwise able to use.

With regard to topping, our conviction is that the best solution is immediately to place a topping of special mix on the mass concrete and finish the slab monolithically.

Mr. Hare. E. HAROLD HARE—I would like to find out what is bedellite, how you would know its presence and if and how you can find it in concrete work.

Mr. Freeman. P. J. FREEMAN—Bedellite is something that a petrographer can pick out. Ordinarily the geologist is not familiar with it.

Mr. Munsell. A. W. MUNSELL—To corroborate Mr. Douglass' experience with the use of a hammer—on the Delaware Bridge and on the piers of the Arthur

Kill bridges from Staten Island to New Jersey, we used air hammers with a speed of about seventeen hundred blows per minute for placing concrete in a position which was inaccessible, that is, in narrow spaces which a man could not enter or where we could not depend on spading or long rods. We secured very satisfactory results with the air hammer.

A. S. Douglass—We are also using hammers quite extensively for general tamping, using the hammer on both the post and the surface of the form. If you use it with discretion and do not knock the flagging off, the results are very satisfactory. **Mr. Douglass.**

Some of the discussions today have mentioned the undesirability of flowing concrete in the forms. Our regular practice in a heavily reinforced beam is to keep dumping the plastic concrete, which will carry its entire ingredients along with it, at one point, preferably at the crossing of four beams. By following it down with a hammer, vibrating the form, the entire mass will slide bodily without segregation. The sliding action wipes out the honeycombing and the results are most satisfactory.

Earl Dutton—I would like to ask if that would not cause a layer of mortar against your forms? **Mr. Dutton.**

A. S. Douglass—We have no evidence of that. It is so effective as a tamping device that the one precaution is not to use it too much. **Mr. Douglass.**

C. B. Foster—I had an experience with air hammers during the war, when in building some boats the engineers in charge insisted on their use. We found that on account of the vibration of the form and the steel, we broke the bond between the partially set concrete and the steel. We also noted that when we stripped the forms great sections of concrete would peel off, and that there was no bond at all under the horizontal steel. In shipping one of the boats from Detroit to Buffalo, it struck a dock with the result that the complete bow was broken off. It would therefore seem that we should use air hammers with caution, especially when the concrete is very dry. **Mr. Foster.**

L. W. Walter—I would like to cite an experience somewhat parallel but decidedly different in the construction of the concrete ship "Atlantis," which was the only one built monolithically and which required slightly over eight days of continuous concreting, operating with two 12-hour shifts. We used air hammers to settle the concrete into place. Concrete was made of a light weight, artificial aggregate, weighing about 118 lb. per cu. ft. Square deformed bars were used, generally with some round deformed bars, offering considerable frictional resistance to the flow of concrete, and I feel that without the use of air hammers it would have been almost impossible to have built that particular ship. The concrete, when it entered the forms, looked rather sluggish, but when air hammers were applied either to the form or to the steel, it would seem to take life, wake up and move along. We had absolutely no trouble from fracturing, due to the application of the air hammers on the steel. The ship was watertight and shows no signs today of any fractures of that nature. **Mr. Walter.**

Kristen Friis—I am indeed very glad to be able to attend this meeting and to have the opportunity on behalf of the Norwegian Society **Mr. Friis.**

of Engineers to express my admiration for the eminent research work carried forward by the American Concrete Institute, the Portland Cement Association and the great number of excellently equipped laboratories connected with the different universities.

Following the remarkable work of Fuller and Thomsen, continued with the not less remarkable work of Prof. Abrams and a number of other scientific brains, it has been necessary for the European engineers to follow closely the research work done in America and Canada, and most of the investigations made today in Europe are more or less based on theories and thoughts coming from America.

When I have the courage to take part in the discussion today, it is not because I can teach you anything but only because I want to lay before you some of our observations in Norway that, perhaps, can be of value in building dams in the northern part of this country and in Canada where the climate conditions are similar.

The Norwegian Concrete Committee was appointed about three years ago by the Norwegian Society of Engineers, particularly to study our concrete dams, because many of them, after 10 to 20 years of service, have shown serious signs of disintegration.

By the examination of the dams, we found that there is up to now no fear of any catastrophe or collapsing of any of the dams due to bad concrete, but that the disintegration is going on steadily and that the leakage through the dams increases with the years. We also found that the trouble begins in the filling places.

In order to find the reasons for this disintegration, we started systematically to make water analyses from 22 different water courses. The tests we made were the pH value; the amount of dry material (alkalinity); the amount of organic material and the temperature. Samples were tested every 14 days for 1½ years to cover the different seasons.

The results showed that the water in all the tested streams is acid water or soft water with a pH value varying from 4.5 up to 6.5 and at the same time very pure, containing very little dry material and also comparatively small amounts of organic material. The average temperature is very low.

As you know, calcium carbonate and bicarbonate are largely soluble in acid and pure water, the solubility increasing as temperature falls. Mr. Bailis, Board of Water Supply, Chicago, has written a very interesting paper about this subject which was given before the Institute of Civil Engineers in 1927.

I do not mean to say that the disintegration of dams is due to acid water, but it must be clear that if such water enters the concrete, the disintegration will be more rapid, and there is no question of self-tightening of the dam. And I mean further that the reason why many of the dams in southern parts of Europe—in Germany, France, Italy and Spain—have shown less disintegration than the dams in Sweden and Norway is due, not only to the better temperature conditions, but also to the water. In making permeability tests in the laboratory this question certainly should be taken into consideration.

Another interesting result of our water analysis is that the analyses of the leakage water not only shows that it is saturated with lime but also that it is absolutely free from organic material. *All organic material contained in the stream water is left behind in the dam.* This organic material will by destruction develop carbonic acid and help in the disintegration of the concrete. There is certainly also the possibility of bacteriological life existing in this organic material. I have gotten that confirmed over here by a test made by Columbia University on samples taken from deteriorated concrete.

It can be easily said that there is no danger from acid water if you only make a dense and absolutely watertight concrete. But it certainly is not so easily done on a large job. I am speaking now of regulation dams exposed to extreme temperature variations.

The water level will vary from full storage down to no storage regularly one or two times a yenr. At the same time you can have extreme temperature variations in the sections of the dam. It can be 40 deg. below zero on the surface and never reach the freezing point one or two feet within the concrete.

Consequently both from evaporation from the surface and from temperature variation in the section you will get surface cracks beginning as hair-cracks. *Exposed to acid water, this hair-crack will develop with the years, because acid water doesn't allow any self-tightening.* On the other hand, if the water is neutral or alkaline from lime or other salts not deteriorative against concrete, we can expect self-tightening and no deterioration.

Under the said conditions with regard to water and temperature, I am of the opinion that it will be necessary, if ordinary portland cement is used, to apply on the surface of a regulation dam a cover with the following qualities:

1. Elastic enough to follow the expected temperature variation without bursting.
2. Absolutely watertight under high pressure.
3. Resistant in water, without getting hard or brittle with the years.

Of course, it cannot be expected that such a material at the same time also shall have the necessary mechanical strength to stand against the rubbing of ice, etc., so it will be necessary to protect it in some way or other. This protection should not be fastened into the concrete by irons penetrating the cover.

As to the material fit for this purpose and covering the qualities mentioned above, I think it is not yet found. When you speak to the coal tar people, you will learn that asphalt is no good in water, and if you speak to the asphalt people, they say the opposite.

Considering that many of the dams in the world need repairs and that such repairs most probably would be best and cheapest done by protecting them with a watertight elastic cover, I think it would be

worth while to make experiments in that line by a neutral and scientific institution.

There are, of course, other ways to be considered, such as making a new cement for that special purpose or by using admixtures to make a denser and more impervious concrete.

As to a new cement the demand for such a special purpose is of course comparatively small and it is understandable that the cement manufacturers, from an economical point of view, are not highly interested. However, it is most probable that a cement standing against acid water also would stand better against sea water, and if these two purposes are seen together, the market is large enough.

Certainly, the best way out would be to make a resistant cement, and I hope that the cement manufacturers have not yet said their last word on this question.

Regarding the value of admixtures advertised to make the concrete resistant against all sorts of destructive agents, I must confess that it has not yet been possible for me to find any strong link between advertisement and actual value.

In most cases I think that a little more cement has given me the same effect. In Sweden they are now selling an admixture containing only very good cement with some hard and fine ground minerals in it. That is, I think, a very good idea.

On the other hand, I will not deny that there is one group of admixture that certainly deserves a very careful and serious study, and that is the materials containing silica oxide in collodial form.

It should especially be of interest to know to what extent the action is chemical and to what extent it is purely physical. According to German investigations, the chemical action depends on the storage of the cement and on the water used in the concrete, because both storage and high water content increase the amount of free lime developed and consequently should increase the demand for silica. On the other hand, it is possible that when silica in collodial form is present, no free lime will develop at all.

And finally, I will only mention the question of construction of the dam. The gravity type dam is far from being the best type to secure a durable concrete and the material in it is not at all economically utilized. In this respect I think both the Ambursen dam and the multiple arch dam are better, because in them it is much easier to avoid surface cracks.

The 600-ft. Concrete Arch Bridge at Brest, France

By E. Freyssinet*

Translated by S. C. Hollister**

I propose to give some facts concerning the present construction of a bridge of reinforced concrete at Plougastel, over the river Elorn, at the point where this river empties into the harbor of Brest, France. The river at this point is 2130 ft. wide at high tide. The tide attains a height of 26 ft. and the current is occasionally very strong.

The minimum opening in a bridge was required to be 564 ft., with a clearance of 118 ft. above a channel in which not even temporary supports were possible. The present design for the bridge was chosen in a competition because of its economy and durability. Altogether much cheaper than any of the competing designs, it permits the carriage of both a highway and a standard gage railway, whereas the other designs provided only for the highway.

A great arch being obligatory for the spanning of the channel, I judged it economical to use for the remainder of the crossing two identical arches in order to re-use the centering. The work then comprises these three arches of reinforced concrete of 612 ft. span center to center of the piers. They support a deck of two stories, the lower of which carries a single track standard gage railway, and the upper a roadway 26.2 ft. wide. At the present time the foundations are completed and in addition the springing of the arches to an elevation 42.5 ft. above high tide; one arch is cast and centers struck while the second is greatly advanced. It is expected that the bridge will be in service some time in 1929.

Abutments and Piers

The arches rest on two abutments and two piers of very low height. Because of the possibility of the decomposition of cement in sea water, concrete using alumina cement called *fondu* was used on that part of these abutments and piers in the tidal range. The proportion chosen comprises 1.79 bbl. of cement per cu. yd. of finished concrete, the aggregate being 0.97 cu. yd. of very hard crushed quartzite found on the site, 0.26 cu. yd. of sand resulting from the crushing of the quartzite and 0.39 cu. yd. of bar sand. To reduce the cost of the alumina cement and to

* Designer and builder of the bridge, Paris, France.
** National Freight and Delivery Company, Philadelphia.

diminish the heating of the massive parts during hardening there was incorporated in the concrete about 50 per cent of quartzite rubblestones.

The foundations of the abutments were accomplished by excavation to a depth of 40 ft. below high tide by means of circular coffer-dams of reinforced concrete 98.4 ft. in diameter and only 11.8 in. thick.

The piers founded at 60 ft. below high water were constructed to the elevation of low water by means of a unique floating caisson (Figs. 1 and 2) of reinforced concrete utilized both as a bell caisson and as a sunken

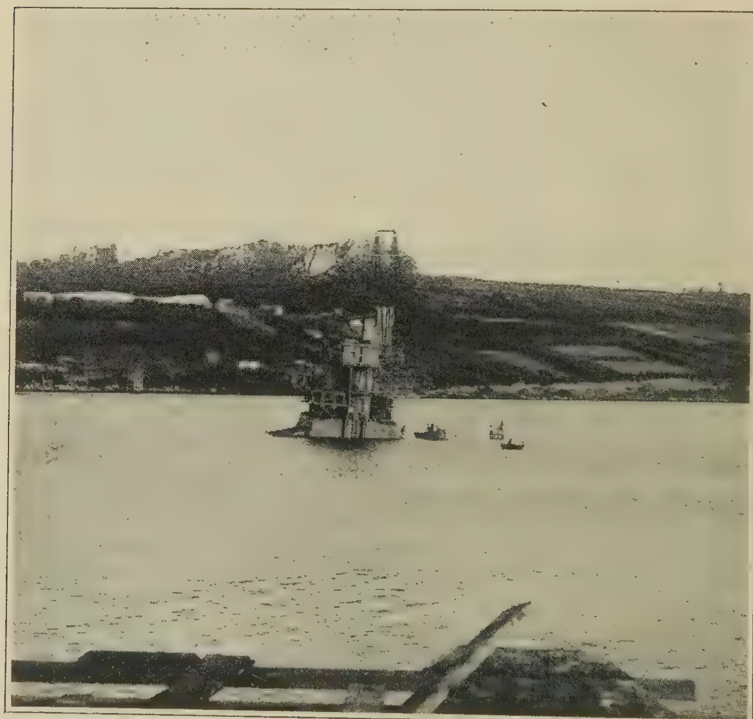

FIG. 1.—SINKING PNEUMATIC CAISSON FOR PIER NO. 2 OF 600 FT. REINFORCED CONCRETE ARCH BRIDGE AT BREST, FRANCE

caisson. This caisson, which was formed to the exact pier dimensions, was anchored in position. Solid ground was reached by increasing the height of the work chamber by successive underpinning with reinforced concrete conducted in such a manner as to enlarge progressively the surface of the bearing of the caisson on the soil in proportion to the extent of the descent, in such a manner as to reduce the unit pressure on the soil and to permit the arrest of the foundation at a moderate depth, whatever the nature of the rock encountered.

ARCHES

The arches, very largely hollow, have a width of 31.1 ft. and a maximum thickness of about 16.5 ft. They are formed by four vertical walls, as may be seen in Fig. 3, binding together the intrados and extrados arch slabs. The thickness of these members increases toward the abutments to about 3 ft. In the central part the total section of the concrete is about a quarter of the area comprised in the exterior dimensions of the cross-section. The form of the neutral axis of the arch is exactly that of the pressure-line of the dead load.

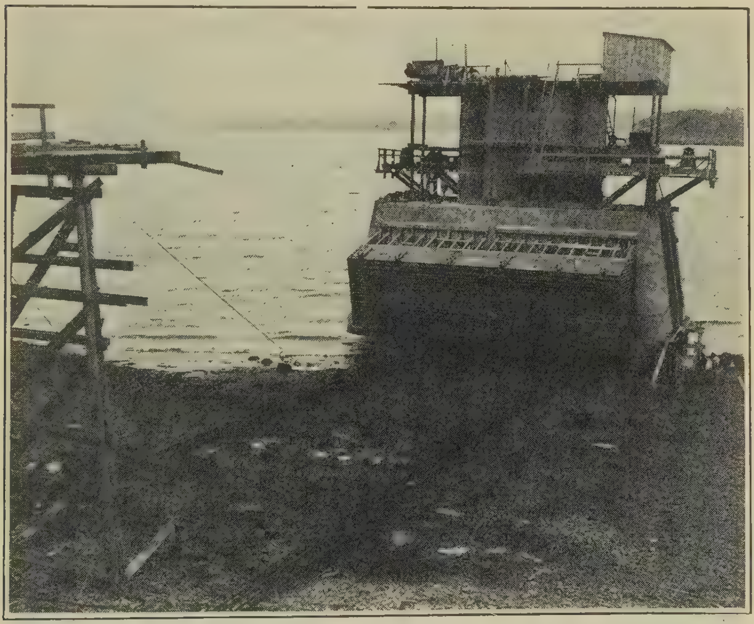

FIG. 2.—LAUNCHING REINFORCED CONCRETE CAISSON.

The central cell is depressed in the vicinity of the crown of the extrados (Fig. 5) to permit the passage of the railway between the side cells. For reasons of appearance and economy, the supports of the deck are spaced 39.4 ft. center to center and the pressure line has a form distinctly polygonal. In order to preserve a good appearance, I have varied the curvature of the arch in such a manner as to obtain a continuous intrados; the polygonal extrados is joined on the sides by the continuous curve of the show-faces.

The calculations are for a fixed arch of 590 ft. span with a rise of 110 ft. and of variable section. One tries to determine as well as possible the secondary or local actions due to the form of the compartments and to

assure an effective distribution of the loads between the different members of the arch. The reinforcement is only useful from the point of view of these secondary actions. The amount of steel employed is very small, approximately 37 lb. per cubic yard of concrete.

FIG. 3.—ARCH CENTERING BEING FLOATED INTO POSITION

FIG. 4.—ARCH CENTERING IN PLACE SUSPENDED FROM HAUNCHES

After striking centers the control of internal tension in the arch will be accomplished by the method which I have described in *Le Genie Civil* of July 30 and Aug. 6 and 13, 1921. To effect this the arches are cut on the plane of the crown by a shallow joint in which are niches to accommodate 28 jacks which will allow elongation of the neutral axis of the arch and thus compensate for the elastic and permanent shortening.

Before performing this operation, however, we are waiting for the results of experiments instituted to determine the amount of this shortening.

The concrete employed for the arches contained on an average 1.9 bbl. of ordinary portland cement per cu. yd. of finished concrete with an aggregate formed of 4 parts crushed quartzite, 1 of sand residue, and 1 of bar sand, and gives strengths which will easily attain 8500 lb. per sq. in. at the time the bridge is placed in service. The total maximum stress in the arches is below 1000 lb. per sq. in., divided as follows: due to arch weight, 453 lb. per sq. in.; due to the weight of the deck, 142 lb. per sq. in.; due to loads of French regulations for highways and trains, 284 lb. per sq. in.; due to linear variations of the arches, 213 lb. per sq. in.

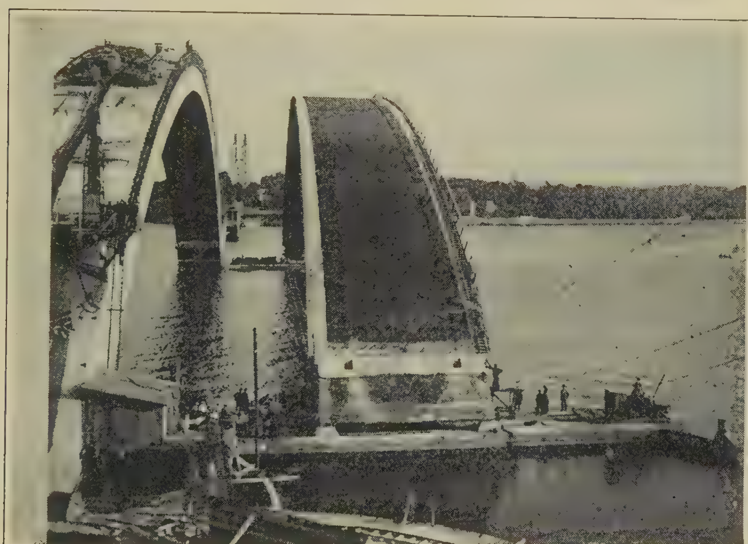

FIG. 5.—REMOVING CENTERING FROM ONE OF THE COMPLETED ARCHES

There will then be a ratio of about 8 between the ultimate strength of the concrete and the stress to which it will be submitted. It is a factor of safety exceptionally high and one which is far from being attained in similar steel structures.

The arches were poured by means of electric cranes on a cable, by a new system constructed by ourselves, of 2260 ft. span on a centering consisting of a wooden arch of the same span as the concrete arches and taking its support on their foundations.

This centering was constructed on the shore and transported to its position of use for the first arch; taken away forty-eight hr. after completing the pouring of that arch and put in place for the second arch,

from where it will be transported to the third arch. Four months elapsed between the first and second moving of the centering.

The volume of wood employed in the centering is 21,189 cu. ft. (254,268 ft. b. m.), or 10 per cent of the total volume of concrete which makes use of it. The stress under dead load of the centering is 142 lb. per sq. in. When the first barrel of the arch is added this stress attains 1000 lb. per sq. in. and increases to 1560 lb. per sq. in. under the total load of the arch. The measured deflections of this wooden arch have been found equal to about 10.4 in. Any permanent deformation could not be discovered.

The second placing of the centering was accomplished in spite of a very strong wind, without the slightest difficulty, in less than 3 hours.

FIG. 6.—ARCH CENTERING IN PLACE AND CABLEWAY SYSTEM

CONCLUSIONS

That which has just been set forth proves simply that the realization of arches of more than 590-ft. span, over an arm of the sea exposed to violent storms and in which one cannot take any support outside of the permanent work, has been accomplished without any feat of strength, and by imposing on the structure maximum stresses very inferior to those generally considered admissible by bidders.

The first important consequence is this—the divergence in essential cost between arches of reinforced concrete and those of steel disclosed by the competition, would have been much greater if we had followed the practice of builders of steel construction in limiting ourselves to the road deck and contenting ourselves with a lower factor of safety.

This raises the question as to what are the limits of span in which reinforced concrete can compete with steel construction? If one varies

the scale of a structure without modifying the design, one causes the stresses due to the dead load to increase in the same ratio as the linear dimensions. The redundant stresses due to the linear variations, contraction and temperature, remain constant. Those due to live load remain likewise constant, if one maintains the same load per square foot on the structure. Consequently, if we double the span of the arches at Plougastel, the computations for such an arch will give us stresses as follows:

	LB. PER SQ. IN.
Stress resulting from dead load of the arches	910
Stress resulting from weight of the deck, about	228
Stress resulting from weight of live loads	285
Stress resulting from linear variations	214
Total	1,637

Under French regulations, a stress of 1637 lb. per sq. in. is legal for concréte testing 5830 lb. per sq. in. at 90 days. This strength is very easily realized by the concrete of the Elorn arches, and it will be easy to better them considerably another time.

Besides, the stresses of the centering under its own load will attain only 285 lb. per sq. in. and it would suffice to double the porportion of the wood section in relation to the section of the barrels of concrete to be supported to maintain constant the stress under the additional load of the concrete.

It is evident, then, that the method of Elorn can be extended without considerable change, to spans in the neighborhood of 1300 ft.

But in the foregoing we have utilized only the strength of concrete without reinforcement for compression or hooping. This is logical so long as the strength is procured at less expense than it would be with steel. But its price increases with the span and after passing a certain limit the metal furnishes the strength more economically. Above this limit it is logical to consider structures in which steel plays the principal role.

Let us consider a span of 5900 ft. and let us maintain constant the ratio between the dead load of the arches and the weight of the deck and that of the live loads—an extremely difficult condition and one from which one departs in practice, the relative weight of the principal girder increasing always with the span.

If the density of the arches were to remain constant, the computation would lead in that case to a unit stress of 9000 lb. per sq. in. If the density of the arch increased in the ratio R the unit stress would be 635 R. It is easy to build members in reinforced concrete capable of resisting compressions equal to 635 R.

Let us consider the arch members built up of square bars electrically welded end to end, separated in the vertical and horizontal directions by transverse bars. One can accomplish perfectly the filling of the interstices between the bars with a rich mortar of fine sand by vibration, and

cover the whole with an envelope of the same mortar well tied by the transverse reinforcement.

One can realize cases in which for a total volume of one cubic yard one would have—

	LB. PER CU. YD.
For the longitudinal steel bars of 2 x 2 in. with clear spacing of 0.4 in. or of 4 x 4 in. with clear spacing of 0.8 in., 7 per cent of the total volume	9,200
For the transverse steel, 3 per cent of total volume	420
For the concrete, 27 per cent of total volume	1,000
Total	10,620

Which gives R = about 36. And $635R$ = 22,800 lb. per sq. in.

Therefore we can let the longitudinal steel be supposed to work alone at 33,000 lb. per sq. in., the mortar being eliminated by its shrinking from all important participation in the permanent stresses. It is a high rate, but nothing opposes its employment in structures of hard steel with a very high elastic limit, the metal not having to submit to any other manipulation than that of electric weldings controllable one at a time and being subjected only to compressions. One would obtain again in this case factors of safety considerably higher than those of any other system of construction. The achievement of such structures composed of members resistant to fire is possible by processes offering a close relationship to those employed at Plougastel.

The conclusion is, therefore, that the arches at Plougastel which at present hold the record for length of span in concrete arches are in truth only very small arches in comparison to those which will be constructed in the near future; and that arches of reinforced concrete, constructed with ordinary portland cement constitute up to the present, thanks to the simplicity of their execution and to the low net cost per unit of strength in the structures, a means for the realization of exceptional spans capable of competing efficiently with all the other systems, without excepting suspension bridges, up to span limits allowable in the present state of metallurgy.

EXPERIMENTS ON CONCRETE CONTRACTION

The variation of contraction of the concrete in relation to the proportions, atmospheric conditions and the mechanical state, has not been made the object of any research which we could utilize for the precise evaluation of conditions of settling of the arches with a view to the suppression of the redundant stresses. We have, therefore, instituted experiments in which specimens of very different proportions have been exposed in the open air to climatic influences under different conditions of stress.

The specimens serving for these tests consist of two plates of concrete 2 x 4 x 71 in., reinforced on one face by three round steel bars of 0.39 in. in diameter. These are stood vertically 4 in. apart with their

reinforced faces opposite each other, and made fast in a block of concrete. The compressions are created by the constant movements resulting from the hooking of weights to brackets fixed to the top of each member. The variations in the separation of the upper ends are measured, this separation being equal, according to the computations of deformation, to the contraction of a member 262 ft. long. The measurements are therefore easily made and can be repeated frequently.

Our reflections lead us to expect an important role of the mechanical stresses in the phenomenon of contraction though in our first experiments, which embraced also the variation of proportions, we have compared only concretes at elastic rest and concretes compressed at 850 lb. per sq. in.

FIG. 7.—FIRST STAGE OF WORK IN CONSTRUCTING WOODEN CENTERING

We have just recently begun again these experiments with loading varying regularly from 0 to 1280 lb. per sq. in. and applied to concretes of different ages, and to concretes unloaded after having been loaded during 20 months. These experiments do not bring forward as yet any new definite knowledge and only confirm in general the first results obtained. They are, however, interesting to know at this time because they throw a light on the phenomenon of deformation of concrete which to many will appear new.

The shrinkage of concrete varies continually with the hygrometrical conditions of the atmosphere. The rapidity of these atmospheric variations is almost of the same order as that of the variations of the interior temperature of the blocks. The proportion in cement between 340 and 1510 lb. per cu. yd. for a volume of the same aggregate of 1350-340-340 has had no important influence either on the speed of the variations or

on the magnitude of the contractions obtained. But while all the test specimens not loaded have undergone very slight shrinkage, annulling themselves in a rainy period, the maxima is between two to three ten-thousandths between a hot and dry period, fluctuating around a very slight value which does not seem to further increase. The variations in the hygrometrical state of the air therefore submit the concrete to alternate shrinking and swelling; but the point in question is not entirely of reversible phenomena. The phenomena of shrinkage are facilitated, and the phenomena of swelling impeded, by the permanent compressions; on the other hand, the swelling seems to be favored and the shrinkage impeded by even very slight tensions.

FIG. 8.—BUILDING UP THE INTRADOS AND EXTRADOS BARRELS OF THE ARCH CENTERING.

In one way or another, under the action of alternate periods of dryness and humidity, the concretes seem susceptible to very extended deformations which they would not have been able to take without rupture under the action of an instantaneous load.

These statements are very important, for one can deduce from them that redundant stresses determined in the arches by contraction are very far from having the importance indicated by elastic computations, since the compressions due to the deformations are attenuated considerably by the exaggeration of the contraction under the action of the same compressions. These things happen as if Young's modulus varied inversely with the deformations to very small values, where the phenomena concerned show bending brought by the contraction.

Concrete reacts and adapts itself to the deformations as a living being. It shrinks locally in order to avoid excessive strains and to carry them back to zones under less strain.

In an arch formed by successive barrels, the first barrels taking more important contractions than the last barrels, the inequalities of pressure between the different barrels tend in the long run to lessen considerably.

These results are important for the construction of large concrete arches and it is desirable that the experiments be made again and carried on in other climates with other cements and other proportions.

CONSTRUCTION PLANT AND METHODS

The handling of the materials often is a very difficult problem in bridges over estuaries, because the condition of the bank uncovered at

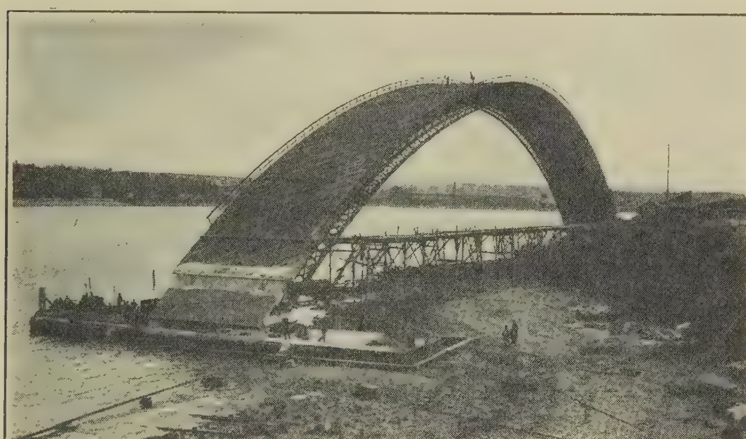

FIG. 9.—COMPLETED CENTERING ON CONCRETE BARGES READY TO LAUNCH.

low tide, the size of the tides and the frequent storms render difficult and burdensome the use of floating equipment. We have had recourse to a conveyor on a cable, capable on the one hand of unloading the boats anchored in the channel, and on the other hand of bringing to and manipulating on the job the loads of reinforcing steel and concrete. The circumstances imposed on this conveyor a span of about 2300 ft.

In general, in these conveyors, the movements are controlled from one of the ends by a series of cables operated by stationary windlasses. In the construction of several large jobs we had discovered that the output of these apparatus applied to the construction of a concrete bridge is very poor and decreases rapidly where the distance from the point of operation of the apparatus and the points of use increases by reason of the variety of services demanded of the apparatus in a work of this kind.

Moreover, in case of a high wind the numerous cables easily become entangled, which impedes the functioning.

The job at Brest is situated exactly opposite the mouth of the harbor and the wind, with velocities at times surpassing 100 feet a second, has a clear sweep over it. We have therefore established a program imposing the following conditions: two apparatus 25 ft. apart; maximum number of cables per apparatus, two; moving trucks carrying a cabin in which a man placed at a good distance from the lifting hook executes with great speed all the maneuvers, under the control of sight and voice. Numerous German, Italian and French specialists were unanimous in declaring that the problem thus posed could not be solved and proposed to us systems which we had condemned in our previous experiments with them. The failure of these specialists forced us to construct the apparatus ourselves, Fig. 6.

FIG. 10.—FLOATING THE CENTERING TO THE BRIDGE SITE.

Each is composed of a cableway of 2260 ft. span between masts, of a weight of 8 lb. per ft. resisting rupture to 231 short tons, stretched under a constant tension of about 66 tons, and regulated by counter weights. The cables are 250 ft. above low tide.

On this cable rolls a truck supported by numerous rollers. This truck is moved by towing by a second cable of ½-in. diameter with a copper core. It carries a control cabin and a 1-ton derrick crane capable of acting upon a tackle-block at 2, 3 or 6 pulley ratios at will. One can accordingly lift a maximum of 4.4 tons. The electric current is brought at 240 volts through the cableway, which is insulated, the return being made through the towing cable. The towing cables of the two apparatus are joined and form an endless cable serving in a pinch to bring the apparatus to the bank. A motor of 15 h.p. runs by a coupling gear with a brake, both the tow boat and the windlass.

These apparatus have run without stopping for almost three years at a speed of 22 to 35 ft. per sec. without accident. They have trans-

ported up to the present time more than 26,000 cu. yd. of concrete and carried out without difficulty all the handling of reinforcement and forms. They maneuver easily even in a high wind and permit a speed and precision of handling which no other system can approach.

Something remains to be said concerning the centering employed for the arches. It is made up of a wooden arch comprising an extrados and an intrados distant about 8.2 ft., joined by lattice work made of single planks. The joists of the extrados are butt-joined and form a continuous wooden arch 32.8 ft. in width and 0.7 ft. in thickness. In the intrados there are sixteen groups of two joists each, corresponding to eight distinct ribs. The butt ends of the successive joists in a single series are stopped 1½ to 2 in. apart, and the interval is filled with rich cement mortar. One secures thus a joint which is incompressible and rigorously adjusted.

The ability of the extrados to hold its shape against the stresses of shearing engendered by the wind is accomplished by the nailing of two continuous rows of planks of a uniform thickness of 0.71 in. following two parallel systems, making an angle of 45 deg. with the axis. A complement of the same system, but in openwork, holds together the eight ribs of the intrados. The whole forms a closed tube extremely rigid in every way, especially to torsion.

No bolts or ties are used in the centering. The work is reduced to strokes of the saw for which no precision is required, and to nailing. The nails used are of the ordinary kind except for pegs without a head 0.4 in. in diameter and 14 in. in length. Eighty-eight tons of nails were used. The framework was constructed for the most part by unskilled labor and on land, profiting by the favorable shape of the bank.

Continuous pieces of the thickness of two joists having the full length of the centering were built and placed on framed bents erected to the desired height. Under their own weight the long pieces took approximately the form of the arch and it was easy to regulate them to the exact form to be constructed.

Setting the Centering.—The first centering was transported to its position of work for the first arch. To this end, we constructed on an incline umbrella sections of the arches extending 52.5 ft. from the centers of the piers; these members were constructed in successive sections with the aid of forms suspended from the sections previously constructed.

I can not enter into the details of the arrangements made to regulate the stresses in the cantilevers and to avoid cracks or subsequent strains in the arches; in a general way we resorted to artificial loading of the centering.

On the cantilevers thus constructed was erected a system of hydraulic jacks to act directly upon a cross beam of concrete from which hangs a sling formed of groups of suspenders ½ in. in diameter.

These slings are terminated on their lower part by a strongly reinforced block of concrete 20 x 22 x 32 in., the lower surface of which forms a point of support for the suspensions of centering.

These slings pass through the arch from top to bottom in holes left through the cantilever sections. There are two of them through each cantilever.

On the other side the centering is terminated at its extremities by a structure in concrete strongly reinforced which can be hooked to the slings by a bolt made up of two pieces each ¼ x 20 in. in cross section and a total height of 3.28 ft. superimposed on an oak block.

I have preferred wood to metal because of its economy and also because it is more deformable; such a cushion of wood can submit without breaking to deformations of 0.08 in. and more.

On this reinforced structure are fastened horizontal screw-jacks acting upon the horizontal cables, which balance the thrust of the centering during its transportation and raising.

These things thus prepared, the barges are brought under the supports of the centering at the building yard; they are run aground on blocks of wood prepared at low tide; the horizontal cables are stretched, the centering is lowered onto the barges by raising it with auxiliary jacks; then the supports for the construction of the centering are pulled down.

At high tide the barges are floated off. Windlasses placed on the barges adjust the centering under the cantilever sections of the piers, their cables attaching to anchorages established at different points in the river. The slings are lowered into the holes, the bolts are placed, the screw-jacks act and the centering is raised so that it fits closely against the intrados of the cantilevers of the arches.

It remains to adjust it to correct the effect of: (1) a possible difference between the real condition and dimensions of the members as compared to the theoretical figures; and (2) the deformations foreseen under loading. For that purpose the centering is brought into contact with and rested upon the cantilevers by a supplementary tension of the suspension slings. One notes the elevation of the crown and compares it to the theoretical elevation obtained by the calculated use of the centering considered as an arch with two hinges.

If one finds a negative difference, for example, one shortens the centering by stretching the horizontal cables, which increases its rise on one hand and permits one to lift up its supports on the other hand.

This regulating enables the contact of the centering with the underside of the intrados of the cantilevers, a condition assured by an extra tension of 88 tons given to each sling beyond the weight of the centering. Also by a padding of cement one submits the centering to moments in the supports equal and in a contrary direction to those of a fixed arch subjected to the extra loads which will be applied to the centering. This is accomplished automatically by the reduction of the tension on the horizontal cables, the position of the points of contact of the arch with the centering having been regulated so that the result should be thus obtained.

Concrete is then poured between the corbel arranged on the cantilevers of the arch and the concrete end of the centering. The block of

concrete thus poured is divided into members by parallel partitions on the symmetrical plane of the arch; a certain number of these are strongly bound with a view to the striking of the centers.

Two days after the completion of pouring one proceeds with the striking of centers, this delay being sufficient for the last concrete poured to attain a strength of 2130 lb. per sq. in.

The removal of the centering is accomplished by demolishing the concrete poured between the centering and the cantilever arms.

The unbound members are removed first; then the bound posts are partially destroyed. By loosening and tightening the slings very small movements of the centering are provoked which bring on a progressive separation, accomplished by a slow crushing of the support of bound concrete which dissipates the energy of deformation stored in the centering.

This energy is far from being negligible, it is on the order of 330 tons, and the striking of centers is one of the most delicate phases of the work, although it presents absolutely no risk, thanks to the precaution taken. As soon as the striking of centers is accomplished, which requires a day, the centering is cleaned and put in place again. The pouring of the first arch being finished the third of August, the centers were struck the fifth of August and the placing of the second arch begun the seventh of August.

Mr. Oliver.

William A. Oliver*—With reference to M. Freyssinet's paper, presented by Mr. A. R. Lord, the writer was very much surprised that no mention was made by M. Freyssinet of what would seem to be a point of great interest and moment in the construction of concrete arches. The writer refers to the use of temporary hinges. The details of this method were presented, September 13, 1926, in a paper given before the Western Society of Engineers by J. F. Brett, entitled, "The Reduction of Deformation Stresses in Fixed Concrete Arches."

In brief the paper states that European engineers in general ascribe their success in building long span arches to the fact that they are erected as three hinged arches and after a period of three or more years when all settlement of structure, all rib shortening due to both compression and plastic flow and all shrinkage of concrete has taken place, the hinges are sealed up and from then on the arch acts as a continuous structure. This eliminates all flexural stresses due to the above named causes and consequently the only flexural stresses that need be provided for are those due to live load. This materially reduces the size of the arch rib which in turn reduces the dead load. In the mean time the structure has been opened to traffic as a three-hinged arch.

While the above may be generally known, the writer feels that it is a part of and should be included with M. Freyssinet's paper.

Mr. Lord.

A. R. Lord (*who presented the paper*)—In talking to Professor Slater this afternoon, he told me that in the California arch dam he had found a direct relation between the temperature inside the concrete and the volume change. M. Freyssinet reports exactly that result, that there was a direct proportion between the shrinkage movement, the volume change in his concrete, and the measurement of the temperature within it. He found that the variation in cement content had very little effect on the volume change.

Prof. Slater.

W. A. Slater—I cannot discuss the paper technically, but I would like to relate something interesting that I ran across not very long ago. As I recall the statement, the span of this arch was 612 ft. I was browsing in the library and ran across a book with which some of you are probably familiar, Hope's *History of Bridges*. It tells of an arch in ancient Babylon across the Euphrates River, 660 ft. long, built of brick. It goes on to tell about a bridge designed to cross the Hudson River, described as the flying pendant lever bridge. It does not give the span of the arch, but there is a scale with the picture that indicates a span of about 1900 ft. This Hudson River arch could be built of wood, stone or cast iron. I offer that as an interesting comment in connection with this very remarkable bridge.

Mr. McMillan.

F. R. McMillan—I do not think we should let the occasion pass without some expression to Professor Freyssinet for this very excellent

* Instructor in Civil Engineering, Case School of Applied Science, Cleveland, Ohio.

paper. I would like to move you that the President be authorized to write M. Freyssinet and say that the paper was very well received and was a distinct contribution to our *Proceedings.*

The motion was seconded by Mr. Hart and unanimously adopted.

R. CLAYE—Mr. President, Gentlemen: It is particularly agreeable to me to see a French engineer applauded by an assembly as distinguished as yours. Very gladly will I transmit to M. Freyssinet your felicitations and I will tell him how much his bold manner of using concrete has attracted and interested you, and how you have admired his beautiful bridge on the Elorn River. **M. Claye.**

Permit me, gentlemen, to take this occasion to tell you how much pleasure I have felt in being in the midst of you to follow your works, which are so scientific and at the same time so practical.

I will carry back from my trip across your beautiful and great country memories of an energetic nation whose constant goal is research for the best in all fields. It is my duty and pleasure to tell you this.

Three and One-half Years Experience of the Detroit Edison Company in Concrete Control

By A. S. Douglass[1] and J. S. Nelles[2]

During the 1925 convention in Chicago, the representative of The Detroit Edison Company was much impressed by a paper on concrete control presented by an engineer of the Pennsylvania Railroad.[3] Two very simple ideas were impressed on his mind; first, excess water as an ingredient of concrete can have no structural value; second, the elimination of excess water increases the structural strength of concrete. From these two ideas it seemed inevitably to follow that quality concrete should pay dividends in the form of cement saving. In other words, the whole program seemed to afford another illustration of what is so frequently encountered, namely, that the producer who for the sake of high quality strictly adheres to the best methods, finds himself financially rewarded thereby. However, as a chain is no stronger than its weakest link, it followed that concrete control must be complete or it would be dangerous.

Accordingly, it was determined that there should be established the most thorough system of control which could be made practicable. It was realized that this could be accomplished only, first by unremitting determination on the part of the boss, and second by the assigning of engineers to execute this control who were of superior abilities and who possessed the force and personality to secure without failure the result necessary, even over the opposition of the "old timers" who had "never done it that way before."

After three and one-half years of conscientious and determined effort and in view of the many difficulties experienced, it is our firm opinion that there are many today who are in the position of talking control and paying for control and not really getting it. The opinion is based on our knowledge of the infinite detail, extreme watchfulness and strong determination which are necessary to accomplish real control, and the belief that few have really awakened to the realization of the difficulties of the task.

However, having worked industriously to secure these results for ourselves during the last three and a half years, we are sure that the money savings accomplished (as will be shown later in this paper), together with the reliability, strength and durability of the finished

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[1] Construction Engineer, Detroit Edison Co., Detroit.
[2] Engineer, Detroit Edison Co., Detroit.
[3] A paper by T. P. Watson, "Concrete Mixtures Under Field Conditions," A. C. I. *Proceedings*, Vol. 21, p. 31.

product, many times warrants the effort which has been given to this matter.

The Detroit Edison Company, therefore, since 1925, has conducted a field laboratory continuously and has proportioned and controlled all

INSPECTION REPORT—CONCRETE RECORD
DRAFTING AND SURVEYING BUREAU
D E CO FORM DS 30

FIG. 1

JOB ______ DATE ______

LOCATION OF POUR ______

AGGREGATE: FROM ______

CEMENT: BRAND ______ FROM ______ ACCEPTED ______

SIEVE ANALYSIS

	WT.	%	WT	%
1½"				
¾"				
⅜"				
#4				
#8				
#14				
#28				
#48				
#100				
FINENESS MODULUS				

COMPR. STRENGTH REQ'D ______

WATER RATIO CURVE A ______ CURVE B ______

GALLONS PER BAG OF CEMENT ______

SLUMP ______

REAL MIX ______ FOR FINENESS MODULUS ______

$r = \frac{mc - m}{mc - mf}$

WT PER CU. FT. DAMP SAND ______ STONE ______ GRAVEL ______

" WHEN DRIED ______ " ______ " ______

" PER CU. FT DRY SAND ______ " ______ " ______

% BY WT. WATER IN SAND ______ " ______ " ______

ABSORPTION OF ______ ______ ______

FREE WATER IN SAND ______ ______ ______

WT PER CU. FT. MIXED AGGR (DRY) ______

RATIO OF MIXED AGGR TO SUM OF SEPARATE VOLS $y = \frac{rwf + (1-r)wc}{wm} =$

NOMINAL MIX ______ BULKING OF SAND ______ GRAVEL ______ FIELD MIX ______

BATCH		A	B	C
CEMENT	BAGS			
TOTAL WATER	GALS			
WATER IN AGGR.	GALS.			
WATER TO BE ADDED	GALS			
GRAVEL	LBS. OR CU. FT.			
STONE	" " " "			
SAND	" " " "			
BATCHES POURED				

SLUMP TEST FROM BATCH ______

" " " ______

" " " ______

SAMPLE NO. ______ FROM ______

" NO ______ ______

" NO ______ ______

TIME IN MIXER ______ ______

HOW PLACED ______

TEMPERATURE ______ WEATHER ______

REMARKS ______

COMPR. STR. AT 28 DAYS OF SAMPLE NO. ______ NO. ______ NO ______

______ APPROVED ______ INSPECTOR

FIG. 1. DAILY CONCRETE INSPECTION REPORT, DETROIT EDISON COMPANY.

its concrete according to the best practice which it was possible to determine. The results of the experience of these three and one-half years of work will be shown in the matter which follows, which indicates an increase in uniformity of quality and an arrival at a point where the

economies resulting from this control are unquestionable. Specifically, it has been reliably proven that for heavy sections, 2000-lb. 28-day concrete can be produced by the use of somewhat less than 4 bags per cu. yd.

COSTS
CONCRETE MATERIALS

NO.__________

JOB ____________________ DATE OF POUR ____________________

CONCRETE DATA:

Strength __________

Slump __________

F.M. of Sand ________ Stone (Pebbles) Gravel (__________ Desired ____________

Real Mix ________________ Shrinkage Factor ____________

Bulking of Sand ____________ Stone (Pebbles) Gravel) ____________

Inundator Used ____________ Field Mix ____________

YIELD:

Cost of Materials per bubic yard of Concrete:

________________ Sacks of cement @ ________________

________________ Cu. Yds. sand @ ________________

________________ Cu.yds pebbles @ ________________

________________ Cu.yds gravel @ ________________

Total. ________________

Remarks:

____

____

____

____

____

FIG. 2. CONCRETE COST SHEET, DETROIT EDISON COMPANY.

of finished concrete. On account of our unwillingness to use as lean a mixture as 4 bags to the yard, where reinforcing or exposure is involved, it has been necessary to set an arbitrary minimum of 5 bags to the yard

for important reinforced exposed work, and it has become necessary in order to take advantage of this amount of cement to increase our minimum designing strengths from 2000 lb. to 2500 lb. at 28 days.

It is the belief of the writers that in the old days of 1:2:4 concrete, the cement content when honestly mixed would run at least 1.5 bbl. to the yard of finished concrete. This was generally calculated to produce a concrete of 2000 lb. at 28 days and such strength was generally used in designs. It is obvious that a practice which with 1.25 barrels to the yard, gives 2500-lb. 28-day strength reliably, justifies an organization of such caliber as to make these results dependable and usable in design.

There follows a narrative of work done and resulting experiences, prepared by the engineer in charge of this work:

AGGREGATES

In April, 1925, the concrete material situation in this district was as follows:

(1) River and lake gravels, not screened.

(2) River sand or fine gravel mixed with crushed limestone and sold as "balanced aggregate" to compete with river gravel.

(3) Pit materials, pebbles and sand, screened and washed, sold separately or combined.

(4) Various grades of crushed limestone.

The best river and lake gravels came from the St. Clair River near its source but a considerable amount also came from Lake Erie. Finer graded gravel and sand came from various sections of the St. Clair River, Lake St. Clair and Lake Erie. This material was low in cost and if carefully selected made excellent concrete, but the grading, as might be expected, was not at all uniform.

The "balanced aggregate" was generally unsatisfactory as it seemed impossible to get the limestone uniformly mixed; the usual method being to mix it on the ground or in a loading hopper with a clamshell bucket, alternating one of sand with two of stone. In addition, the limestone ran much to one size and in most cases was badly coated with crusher dust, while the sand generally ran too fine.

The pit materials were used on all jobs where the material could be brought direct to the mixing plant by carload, and were invariably bought as separated aggregates. There are a number of these pits in operation and in general they turn out good concrete-making aggregates. Some of the pits have better facilities than others for grading their product but owing to the fact that most of them have an over-run in the size ranging from No. 4 sieve to the ½ in., it is rather difficult to get an aggregate that does not run too heavy in this size.

During 1926 one of the pits set up a laboratory and did considerable work in the way of controlling the mixing of the aggregates on the loading belt. Other pits followed suit and during 1926 and 1927 a great deal of

material was shipped that was consistent as to grading from car to car. However, beginning with the fall rush in 1927 when the supply men were rushing in their winter stocks, there was a decided falling off in this respect which has continued during 1928 and was particularly bad during the fall just past. It seems to have been caused by a greatly increased demand for concrete aggregates, particularly during the last few months, when the pits apparently had no difficulty in disposing of their output. Under such conditions it is difficult to expect a producer to change his

FIG. 3. BOTTOMLESS BOX PLACED IN MIXER SKIP FOR MEASURING GRAVEL.

loading system, thereby slowing up his production in order to give a particular customer a more carefully graded material.

The fault seems to be that the customers who are using control methods are a few large users. The smaller users are not so discriminating and they in the end use the most of the material. It is more difficult to get good consistent gradings from the supply yards by truck than it is from the pits.

One can change the design of the mix to suit different gradings during the progress of a job, but it is not practical to make these changes from batch to batch on one pour, and therefore a consistent grading is a necessary step in quality control.

A great deal of ready-mixed gravel was, and still is, shipped to the local supply yards from the pits, to compete with the mixed river and balanced aggregates already mentioned; and is retailed at a considerably lower price than it is possible to buy the equivalent in separated aggregates. This tends to retard the general use of separated aggregates in this district.

In general, the crushed limestone is lacking in grading and in addition has in most cases a very objectionable coating of crusher dust. We have made many tests and find that these coatings do not come off. Stones picked out of the forms after concrete is placed reveal that the coatings are not penetrated by the cement. This may not be all crusher dust; much of it no doubt is caused by the rubbing of the particles together in handling.

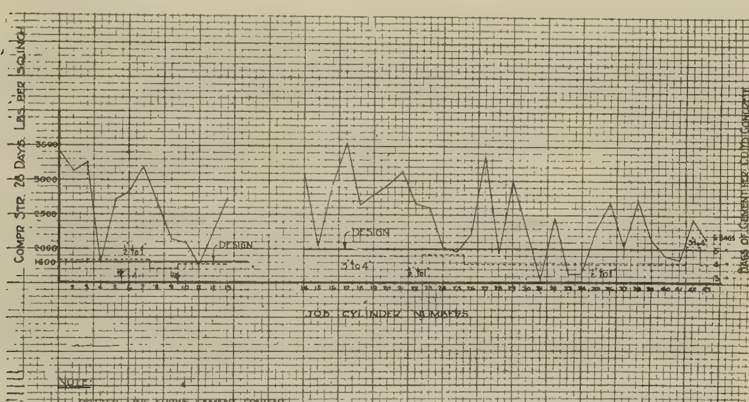


FIG. 4. GRAPHICAL ANALYSIS OF TEST CYLINDER BREAKS, NORTHEAST SWITCHING STATION.

Early Experiments.—So far as we know there was no water control in effective operation in this district prior to 1925.

After two or three weeks of intensive work with sieve analyses, bulking tests, slumps and mix designs it at once became evident that a daily inspection report, to include all the design steps in their proper order, was essential, and the one shown in Fig. 1 was developed. It has served our purpose for the past three years, and in addition to being a valuable aid to the inspector it has proved to be a very satisfactory record, particularly in the case of remodeling or revision jobs where the job conditions at the time of the pour are of great value to the designing engineer. A few months later this was supplemented by a cost sheet as shown in Fig. 2.

FIRST CONTROL JOB

We were then ready to put control methods into effect and selected a heating plant caisson foundation job to start on. Owing to the fact

that it was a downtown job, with cramped facilities, it was decided to use a ready-mixed gravel, especially as good river gravel could be had at a low cost. The grading of this material was checked by sieve analysis daily, also the moisture content and bulking, and the quantities of cement and aggregate were adjusted. Frequent water adjustments were also found to be necessary in order to hold to a uniform consistency.

The method of placing the concrete in these deep caissons was to buggy it to a small hopper centered over each caisson. From the opening in the bottom of the hopper a square wooden chute was carried down vertically only 2 feet. From this the concrete dropped free to the bottom of the caisson. Had the concrete not been workable and of extremely uniform consistency due to the control, this would not have been possible without segregation.

Each caisson was inspected carefully during the progress of the pour, the inspector going down into each shaft at least 4 or 5 times. In some cases it was found necessary to straighten up some of the reinforcing steel, but at no time was there found any segregation in the concrete.

The data on this job follows:

Equipment

Tilting mixer, ¾ cu. yd. capacity, fed with skip.

Water tank of rectangular section with an overflow pipe adjusted to any level by means of an elbow connection.

A bottomless box made to fit into the skip, into which the gravel was shovelled. When filled to desired level the box (Fig. 3) was removed. It will be noted that the sides of the box are made on a batter to facilitate its removal.

Job Data

Number of caissons	15
Diameter of shaft	5 ft. 7 in.
Diameter of bell	14 to 16 ft.
Depth	86 ft. (average)
Design strength of concrete (Curve B)	3500 lb. per sq. in.
Cement per cu. yd. of concrete	7 bags
Maximum size of aggregate	2½ in.
Slumps	1 to 2 in.
Number of test cylinders	52
Average break	3939 lb. per sq. in.
Variation per sq. in	2285 to 6765 lb.

Undoubtedly some of the low breaks were influenced by the large aggregate as no attempt was made to eliminate the larger stones from the cylinders.

One very definite advantage gained from this first experience was a much greater familiarity with concrete aggregates, both as to their concrete making qualities and also their cost per yard of finished concrete.

Other important conclusions were:

(1) That where ready-mixed aggregate is used too much care cannot be exercised in getting the sample of the gravel for analysis. A generous

amount, representative of the stock on hand, should be taken and quartered down to proper size for analyzing.

(2) That for average reinforced concrete work the sand ratio should be greater than 33 per cent if a desirable degree of workability is to be obtained.

EXTENDED CONTROL

The above data looked so encouraging in spite of the great variation in the test results, that it was decided that control methods would be extended to include all The Detroit Edison Company concrete work.

FIG. 5. THE FIRST TWO TRANSFORMER MATS POURED AT NORTHEAST SWITCHING STATION.

The field in general was skeptical but very willing to cooperate. Without this cooperation, it would be difficult indeed to get satisfactory results.

We very quickly found that it was much better to show a foreman or a workman why he should not do a thing rather than threaten him if he did it again. To this end we demonstrated the simpler tests to the men on the concrete crews at every opportunity. A demonstration of the bulking test alone made us many friends among the workmen as it proved to them that the old arbitrary method of measuring the materials was wrong and that seven shovels full of sand did not always make a cubic foot.

During the balance of 1925 control methods were carried out on a dozen other jobs of various magnitudes. No new equipment was purchased but water measuring tanks were made in our own shop. These tanks were gradually improved both for speed and accuracy and the latest model will be described later. The mixers varied in size from ¼ to 1-yd. capacity, the smaller ones equipped with skip loaders and larger ones with material hoppers.

The methods of measuring the aggregates varied on different jobs. The ingenuity of the superintendents and concrete foremen was called

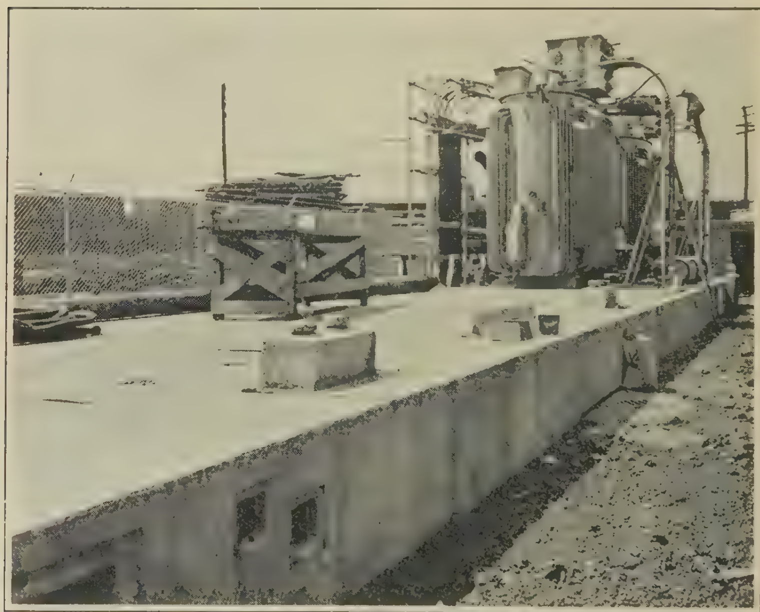


FIG. 6. ONE OF THE RECENT TRANSFORMER MATS CONSTRUCTED AT NORTHEAST SWITCHING PLANT.

into play and resulted in some very practical ideas, especially with reference to measuring materials in skips.

In November, 1925, we purchased an inundator. We experienced the following difficulties with it during the breaking-in period:

(1) With the inundator adjusted at its minimum capacity we had to run approximately 1⅛ cu. yd. to the batch, although it was intended for use with a 1-cu. yd. mixer. However, the mixer did not slop over and after strengthening the skip hoist guide angles, we were able to handle it.

(2) It was not fool proof, and required an intelligent, conscientious workman to handle it. For instance; it was possible to

fill it in half a minute, or to take as long as two minutes, and the amount of inundation obtained would vary greatly in the two extremes. It was found that the filling time should be held consistently at about one minute.

(3) We found that invariably we had to add less water direct to the mixer than our design showed. After a careful study of this, together with innumerable tests in the laboratory to ascertain whether we could get 100 per cent inundation or not, we finally concluded that 95 per cent was about the limit. On checking the inundator by drying out and measuring the inundated sand we

FIG. 7. BOTTOM-DUMP BATCH BUCKET HAULED BY ELECTRIC LOCOMOTIVES AT DELRAY POWER PLANT NO. 3

found it checked the 95 per cent previously determined in the laboratory. This is not a detriment to the device as long as it is understood and taken into account. It is advisable, however, to determine the correct percentage on each job.

This then was the answer to the water difficulty, *i.e.*, in addition to the water carried in the inundated sand approximately 5 per cent of the volume of the inundator was all water.

From this it will be seen that the inundator is not an absolute water measuring device. However, it does measure a uniform amount of sand to the mixer when properly handled and is therefore an aid to good concrete. We have since purchased another one of a larger size.

Analysis of Costs 1925

From the records of concrete poured from April to December 1925 the following data was tabulated:

Table No. 1 shows the variation in the material cost per cu. yd. of concrete due to all causes.

Table No. 2 shows what proportion of this variation in material cost is due to the grading of the gravel supplied and should impress the necessity for superintendents to refuse to accept delivery of gravel low in coarse aggregate.

Table No. 3 shows what proportion of this variation in material cost is due to the use of high slumps and should be a guide for the superintendent in determining how many extra laborers he can afford to use, or what extra expense he can go to in order to properly place concrete of lower slump and still save money to the job.

Table No. 4 shows what bearing the bulking of the aggregates has on the cost of a cubic yard of concrete. The natural conclusion would be to stock up with material on dry days and avoid deliveries on wet days, but unfortunately other considerations enter into the problem especially on truck delivery jobs, and so the table is of more interest than practical value.

Table No. 5 furnishes a comparison between the material costs per cu. yd. of concrete using river gravel or separated materials. Having due regard for the local prices of materials it should be a guide as to which material is the most economical.

In all these tables the yield has been assumed to equal the real mix, which is conservative, especially in the case of the higher strength concretes.

With the exception of No. 5 these tables all deal with river gravel or ready-mixed gravel, only.

During the latter part of 1925 river gravel of a good grading was becoming more difficult to get and in the Spring of 1926 more of the ready-mixed material was being shipped from the pits. It was, however, difficult to get the pits to put enough coarse aggregate in the mix at that time, and the proportion of fine to coarse was more likely to be about 50–50. During 1927 this condition was more or less reversed and we were getting too much coarse aggregate in the mixture. Some pits, of course, were better than others in this respect.

Handling of Aggregates

About this time we began to pay more attention to the handling and stocking of the materials on our jobs, trying, in so far as possible, to get away from segregation. Instructions were issued to field men, when unloading concrete aggregates to lay the material down in layers rather than peaking the piles, but the evil was not entirely overcome and is still a problem because the materials come badly segregated in the cars.

We have studied the loading methods at the pits and believe that the difficulty is not impossible of correction.

Invariably the material is either brought up to a small loading hopper on a belt conveyor, or loaded into cars direct from large storage hoppers. In both cases it is delivered to the cars through a closed-in chute hung over the center of the car. The end of this chute is about 7 or 8 ft. above the bottom of the car and in many cases it is allowed to remain stationary producing a series of peaked-up piles in the car. If the chute could be brought down lower and automatically swung from side to side so as to spread the material, while the car is moved shorter distances and more often, we believe there would be less segregation.

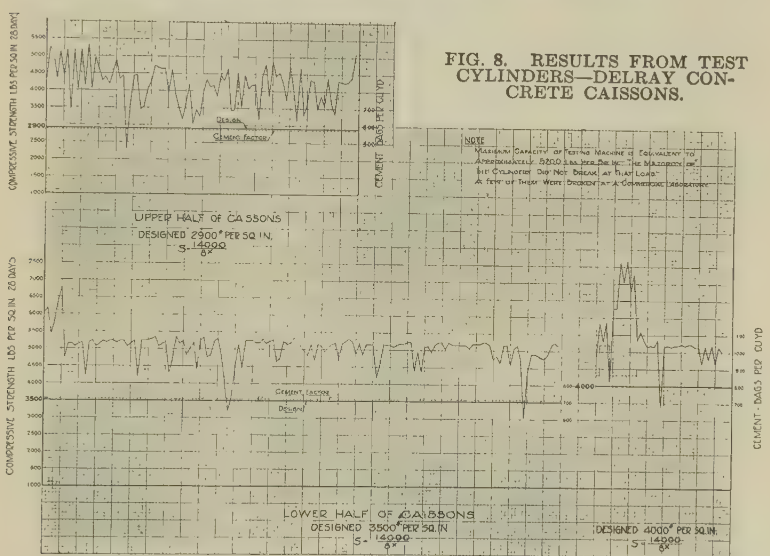


FIG. 8. RESULTS FROM TEST CYLINDERS—DELRAY CONCRETE CAISSONS.

There would also seem to be much room for improvement in the handling of aggregates in the local yards. The prevalent practice is to clam the material out of the cars and dump it into huge peaked-up stock piles. In many cases the piles are so big that the coarse material rolls down the slope and eventually reaches a point far out of reach of the clam. Naturally a great deal of segregation occurs here.

This is not a difficult fault to overcome nor will it cost the supply man more to have it taken care of, and certainly the informed customer will show his appreciation through patronage.

The ideal supply yard of course would stock the aggregates in several different sizes, in such a way as to be able to assemble on a belt any given combination of those sizes that might be required, and deliver the aggregate to the customer well-mixed and with a minimum of segregation.

Another question that the supply man might well study is "How

much is a 5 or 6-yd. load?" Our experience is that the actual amount of material delivered from load to load varies to a surprising extent.

VARIATION IN CONSISTENCY

Up to this time we found that no matter how carefully we figured our water content in the aggregates we still had to make frequent adjustments to the amount of water added to the mix in order to get the desirable uniformity in the consistency of the concrete. Uniform strength we know is much to be desired but in our opinion uniform consistency is also very

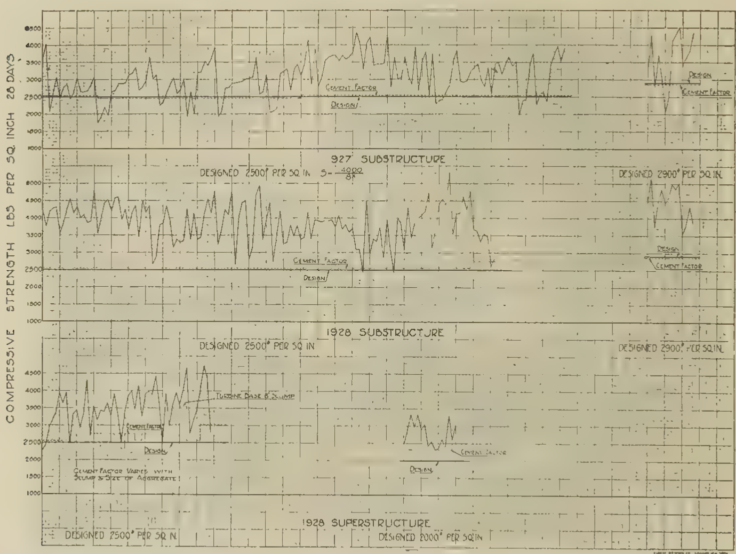
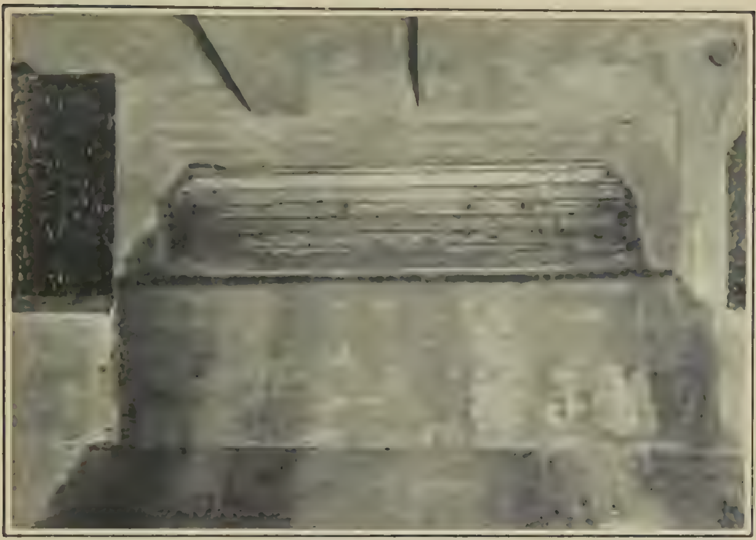


FIG. 9. RESULTS FROM TEST CYLINDERS—DELRAY SUBSTRUCTURE AND SUPERSTRUCTURE.

important particularly if laitance is to be eliminated. After checking the weights of some 270 bags of cement on the jobs at various points in this district, we soon realized that this difficulty was due to the great variation in the amount of cement in a bag. Naturally if the same water-cement ratio is maintained from batch to batch the compressive strengths should be uniform but if the same amount of aggregate is added to this varying cement paste, the consistency will not be uniform.

The results of these tests are tabulated in Table No. 6. There is room for study here, as such a variation is not necessary. If these tests are a criterion of the general practice it is clear that the mills can help the field to obtain a more uniform concrete and at the same time make a net saving to themselves of about 2 per cent of the cement bagged.



FIGS. 10 AND 11. CONCRETE WORK IN DELRAY POWER HOUSE.
Fig. 10, above, shows a turbine overflow and Fig. 11 a part of the overflow canal.

LOW CEMENT CONTENT

As our control methods developed we found that we could reduce and still further reduce the amount of cement necessary to get a good 2000-lb. concrete and to this end the slumps were kept at a minimum. This resulted in very considerable savings in cement. The Northeast Switching Station job is a typical example of this and so will be given in detail:

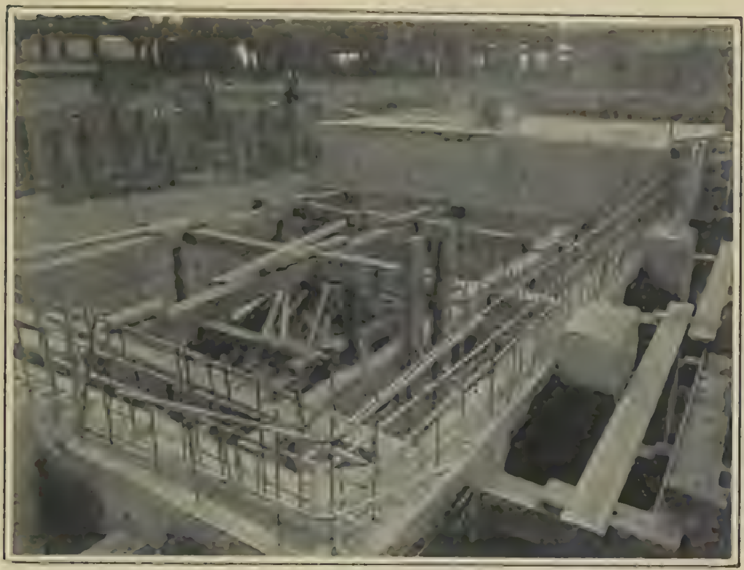
Job Data

Total cu. yd. of concrete	954
Total bags of cement used	3901
Bags cement per cu. yd. finished concrete	4.09
Average material cost per cu. yd. finished concrete	$4.67
Total number test cylinders taken	43
Test cylinder data (See Fig. 4)	

It will be noted that the average cement content for the whole job was only 4.09 bags per cu. yd. of concrete and that the compressive strengths varied from 1600 lb. per sq. in. to 3500 lb. per sq. in. This variation is excessive but the control of the minimum breaks is very good, there being only 4 cylinders that fell below the design. No doubt much of the variation in cylinder strengths is due to the excessively low slumps. It is difficult to make good cylinders with such dry concrete, but in footings and mats such as these, where the concrete can be tramped on and rammed it is probable that the variation would not be as bad in the job concrete. This job had no special equipment except the water measuring tank of our own design on the mixer. Fig. 5 shows the first two transformer mats poured and Fig. 6 a more recent addition to the job.

During the progress of the Beacon Heating Plant job we had an opportunity to try out some interesting experiments with low cement contents. Owing to the fact that it was only the first section of this plant, there were two temporary common tile walls on the two sides where the extensions would eventually be built. On each of the floors on those sides there was a strip of temporary slab about 12 to 18 in. wide extending from the girders over to the temporary walls. We experimented with these slabs and got the cement content down to 3.00 bags per cu. yd. of finished concrete. The data on the two lowest cement contents is given below.

Date	April 16, 1926	May 3, 1926
Material	Gravel of F. M. 5.3	Gravel of F. M. 5.9
Water	9.3 gal. per bag	9.4 gal. per bag
Slump (actual)	1 to 2 in.	1 to 2 in.
Field Mix	1:9.4	1:10.8
Cement Content	3.25 bags per cu. yd.	3.00 bags per cu. yd.
Time of Mix	2¼ to 4 min.	1½ min. for 1st cyl. 10 min. for other two
Test Cylinders	1600 lb. per sq. in.	1640 lb. per sq. in.
	1860 lb. per sq. in.	
	1730 lb. per sq. in.	2570 lb. per sq. in.
	1840 lb. per sq. in.	2740 lb. per sq. in.



FIGS. 12 AND 13. CONCRETE WORK IN DELRAY POWER HOUSE.

Fig. 12, above, shows a sump and oil storage pit and Fig. 13 a portion of a coal unloading house.

It is unquestionably true that the low slump and the time of mixing has helped a great deal and the extremely high strengths obtained in the batches mixed for 10 minutes is of particular interest.

About a year and a half later these slabs were broken out and the concrete was carefully examined. A visual inspection did not show a very great degree of porosity and it seemed to require about the same amount of work to break them out as it did for the others of greater cement content.

This was merely an experiment and it is not to be inferred that reinforced slabs should be poured with such a low cement content. This matter of minimum cement content will be referred to again in this paper.

1926 Summary.—Table No. 7 is a summary of the 1926 concrete work with an analysis of the test cylinders. It shows that seven jobs were completed with an average cement content of 5 bags or less per cu. yd. of finished concrete and that the grand average cement content for the year was 5.16 bags. This covers all classes of work and represents a very considerable saving in cement, in addition to having a more uniform concrete of better quality.

ADMIXTURES

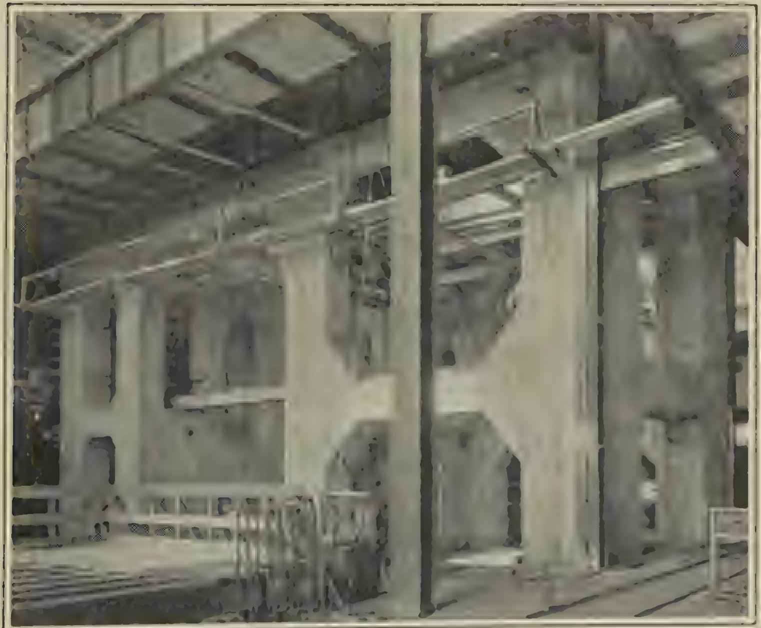
About this time the admixtures began to come to our notice and we tried out a few of the most likely ones. We found that some of them had a certain amount of merit but when used on a controlled job did not warrant the extra expense. Unquestionably on jobs where arbitrary mixes and no water control are still the rule, or where the aggregates used are lacking in fines, the use of some of the admixtures will improve the workability and strength of the product.

We have since experimented in our field laboratory in this connection and a complete tabulation of the tests appears in Table No. 8.

The procedure in making these tests was as follows: Twenty cylinders were made from each batch, and 4 of them were broken at each of the following periods: 7 days, 28 days, 60 days, 4 months and 6 months. All of the cylinders were cured in damp sand until broken, up to 60 days and left in air, at room temperature, for the balance of the six months. The yield was checked by rodding the concrete into a box made just about the right size to accommodate the batch.

It will be noticed that the water-cement ratio is not maintained uniform through the different batches. We had intended doing this but found that the admixtures had such a definite bearing on the consistency of the mix that in some cases it would not have been possible to get the concrete into the cylinders. We, therefore, endeavored to hold each batch to the same consistency by judgment, checked by slumps. The difference in workability between the various batches was not enough to cause comment.

The material cost per cu. yd. of concrete, based on the actual yield, is also given and in two cases comparative batches were made by adding



FIGS. 14 AND 15. CONCRETE WORK IN DELRAY POWER HOUSE.

Fig. 14, above, shows foundations for a screen house and Fig. 15 a turbine base before finishing.

cement, equivalent in value to the cost of admixture, in direct comparison with the batch containing the admixture.

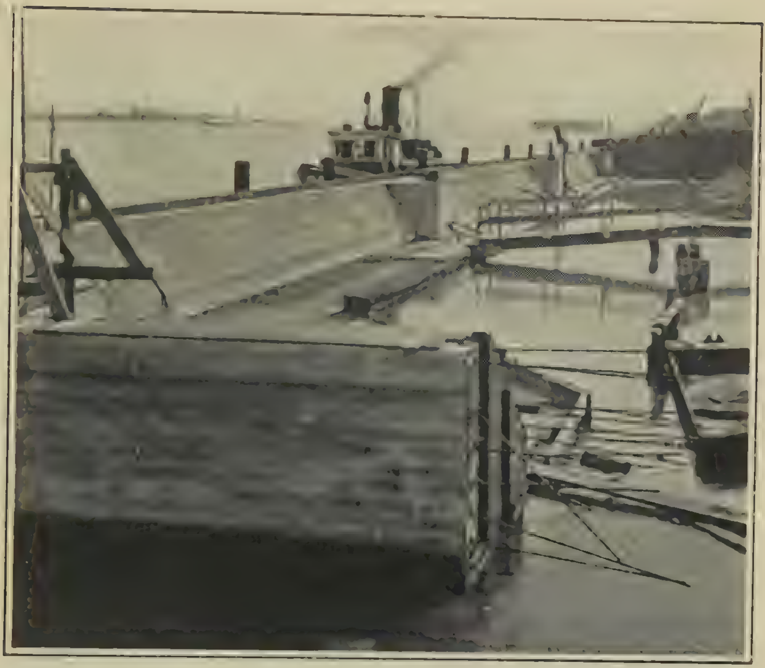
It would seem that cement is about the best admixture, whether considered from standpoint of strength, workability, or convenience of handling, and we are of the opinion that the value of other admixtures, in good concrete, is subject to much question. We realize in making this statement that there are many who will be disinclined to accept it, and many who honestly believe that they have extensive, carefully secured results which will disprove it, but it is, nevertheless, our honest conviction from such work as we have been able to do. Emphasis should be placed on the fact that these remarks are made only with respect to carefully prepared concrete.

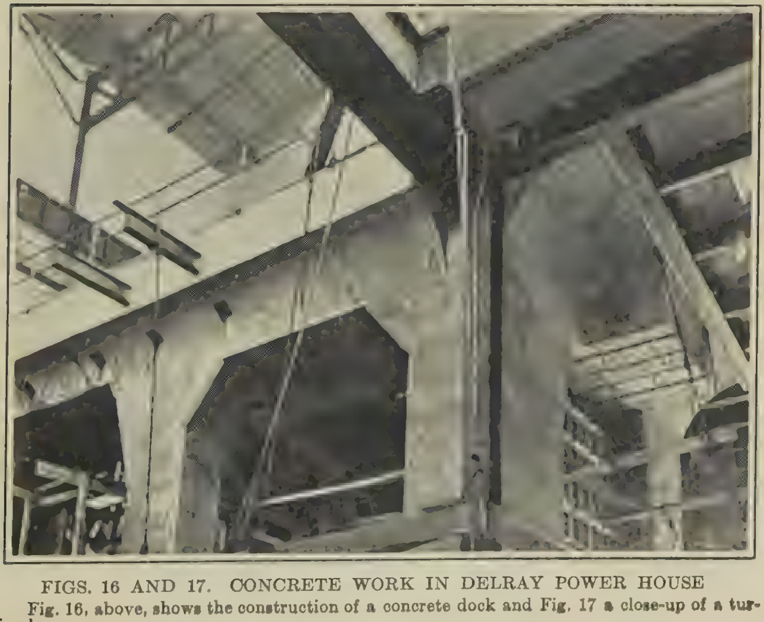
USE OF CHUTES

Control concrete does not necessarily do away with chuting concrete into place. We do, however, insist that the slope of chutes shall be 2:1 throughout their entire length, and that the chutes be so placed as to get the concrete to all parts of the forms without the necessity for excessive flowing, hoeing or shovelling into place. Our experience has been that it is much better to place the concrete by the use of buggies; batch buckets handled by locomotive crane; or a combination of the two methods. With these methods the concrete can easily be placed in every part of the form without the necessity of shovelling. In connection with the buggied jobs a single length of chute from the tower to a receiving hopper is often used to advantage. On such jobs we have found that there is a tendency to get the slope of the chute too steep which allows the coarse aggregate to segregate badly. In our opinion the 2:1 slope is about right.

DELRAY POWER PLANT NO. 3

In the fall of 1926 we broke ground for the first section of a new power plant at Delray, the foundations of which were to be caissons carried to rock about 90 ft. below ground level. As there were to be over 100 of these caissons containing 8600 cu. yd. of concrete in addition to some 24,000 cu. yd. in the substructure and superstructure, considerable thought was given to the concrete equipment to be used. It was finally decided to install an inundator in conjunction with a volumetric stone batcher, a cement weighing batcher and a suitable water-measuring tank for the excess water. We increased the size of the equipment to a 1½ yd. batch capacity to enable us to hold the batch in the mixer for a minimum of 1½ minutes. The concrete was handled from the mixer to the job in batch buckets of bottom dump type transported on specially built trucks travelling on narrow gauge track as shown in Fig. 7. In most cases the bucket was swung over the form by locomotive crane and dumped in the precise place it was required. In places out of reach of the crane, the buckets were dumped into a hopper and the concrete buggied into place.





FIGS. 16 AND 17. CONCRETE WORK IN DELRAY POWER HOUSE
Fig. 16, above, shows the construction of a concrete dock and Fig. 17 a close-up of a turbine base.

The pebbles and sand were stocked in large quantities, and during the delivery period many analyses were made. The size of pebbles used was 2½-in. maximum size for caissons and substructure and 1-in. maximum size for floor slabs and superstructure. The final mixes were designed on the basis of an average of all the tests. It was found that these designs gave us a very uniform concrete throughout the job. As we got down to the bottom of the stock piles there was sometimes a

FIG. 18. ADJUSTABLE STEEL GATE FOR ACCURATE BATCH MEASURING IN THE SKIP.

slight change in the workability of the mix, but with a little attention given to careful loading of the cars going to the mixing plant, it was generally taken care of. When fresh supplies of materials were received many analyses were made and the mix adjusted to suit the new material. In the winter work some difficulty was experienced due to excessive moisture content in thawed out pebbles, to correct which complete inundation of the sand had to be sacrificed to some extent. However, the percentage of sand not inundated was small and consequently the

bulking of this small percentage did not appreciably affect the uniformity of the concrete.

Table No. 9 supplemented by Figs. 8 and 9 give the job concrete data. The saving in cement is quite evident from these figures. Although over 5000 cu. yd. were poured using more than 7 bags per yd., the grand average for the job is less than 6 bags. It is interesting to note also that we held to a 5-bag minimum for the substructure and superstructure as

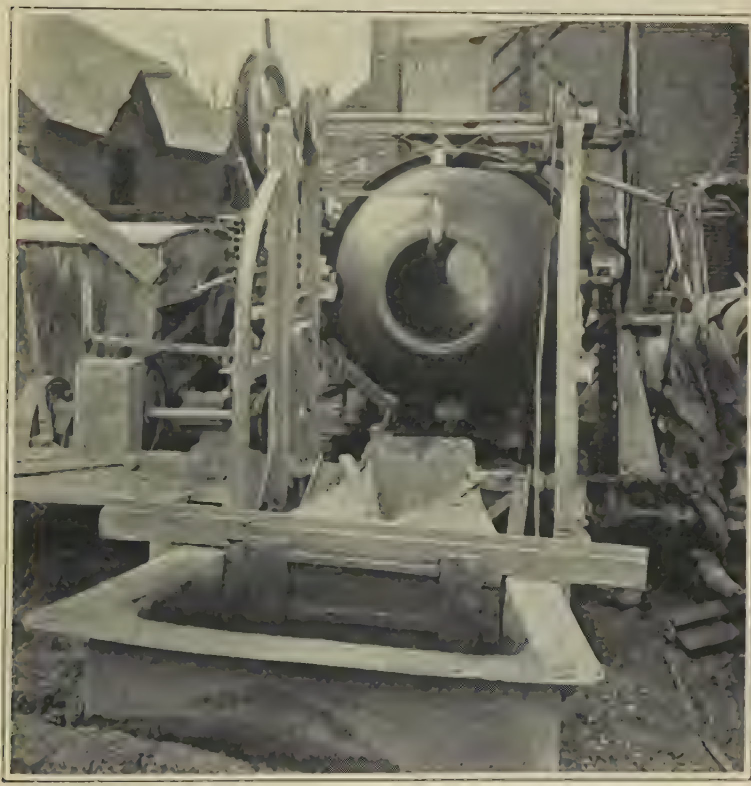


FIG. 19. WOODEN REMOVABLE PARTITION FOR ACCURATE BATCH MEASURING IN THE SKIP.

we did not care to carry the economy below that point on this job. For the caisson concrete we held to a $5\frac{3}{4}$-bag minimum with 7 or more bags per yd. in the bottom half. This latter precaution was taken on account of the presence of sulphur water in the rock seams.

From this data it will also be noticed that there is considerable variation in the cylinder breaks, but the minimum is usually above the design. During the first month on the substructure work we had some

12 cylinders break below the design but this was found to be due to occasional over-sanding when the crane operator loaded the sand hopper too full and allowed it to overflow into the stone hopper.

On all the caisson work the following method was adopted to place the concrete. A hopper was lowered to a point about 25 ft. below the ground level and centered over the shaft. The locomotive crane lowered the batch bucket to this point, and the concrete passed through the hopper into a couple of sections of 8-in. circular chute from which it dropped free to the rock at the bottom of the caisson. On account of this method of placing, the sand ratio was increased to 45 per cent of the total separated aggregates as it was found that this mix held together better. The caissons were poured at a 3 to 4-in. slump.

In the substructure and superstructure concrete we varied the sand ratio from 40 per cent when combined with 2½-in. maximum size aggregate to 45 per cent when the 1-in. maximum size material was used.

Figs. 10 to 17 inclusive illustrate the class of concrete work on this Delray Power House job and give some idea at least of the general appearance of the finished product.

The concrete went into place without much difficulty and very little honeycomb was found, due first of all to the fact that no concrete was poured at lower slump than 3 or 4 in. and in the case of slabs a minimum slump of 5 in.; and second, to the use of a pneumatic tamping tool developed on the job. Care must be exercised in using such tools excessively as the tendency is to pull the mortar away from the aggregate. In our work the reinforcing is heavy and in the floor slabs there is generally an excessive amount of electrical conduits, and this class of work requires such treatment, but it can be overdone if not carefully watched.

The lack of honeycomb on the Delray job was the direct result of trouble we encountered in the fall and winter of 1926 and 1927. After two years work, designing 2000-lb. per-sq.-in. concrete and stressing low slumps we began to get excessive honeycomb. The causes were:

(1) We were running to the extreme in dry mixes.

(2) The drier the mix the harder it was to get the average laborer to puddle the concrete properly.

(3) The tendency for the foreman to leave too much to the inspector. He had been instructed that the inspector was to have full authority in the matter of mixes and figured therefore that it was not his responsibility if the product was full of honeycomb. We had his cooperation but had probably taken too much supervision away from him. This is a point to be guarded against. It is necessary that the inspector should have authority but the foreman must understand that he is responsible for satisfactory placing.

1927 Summary.—Table No. 10 is a summary of the 1927 concrete work with an analysis of the test cylinder results. It will be noted that if we exclude the Delray caissons there was a total of over 27,000 cu. yd. poured with an average slightly less than 5 bags of cement to the yard.

SMALL JOB CONTROL

Concrete control no doubt appeals to many engineers as a thing to be considered in connection with jobs of considerable magnitude only, jobs that will stand the expense of special equipment and supervision, but we find that control can be successfully applied to all classes of jobs, big and little; but the manner of applying it changes somewhat.

At the start of all jobs recommendations are made to the superintendent and purchasing department as to the source of supply of materials. If possible a sample of the material is brought to the laboratory for

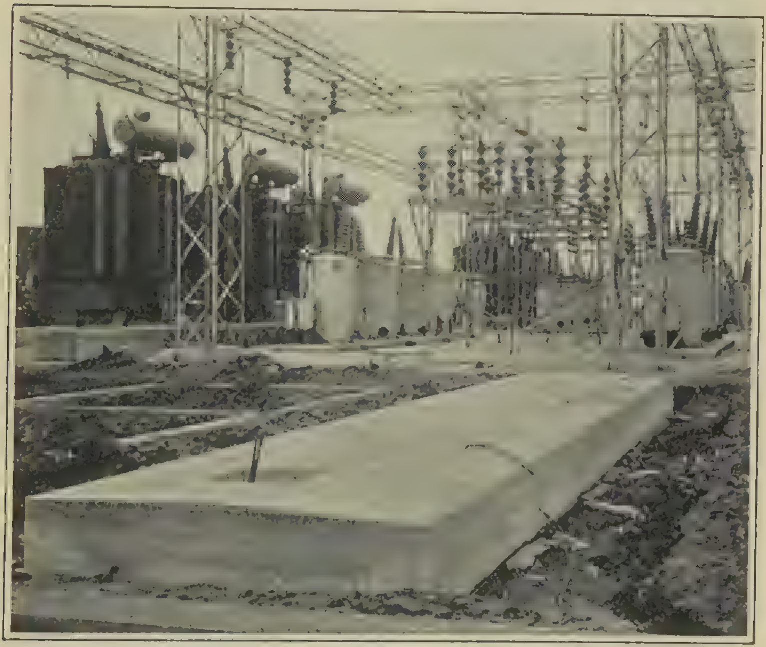


FIG. 20. TYPICAL SMALL TRANSFORMER MAT.

analysis and the mix designed. In many cases on the small jobs the inspector has not the time to get a sample of the aggregate into the laboratory for analysis before the pour, and so has to work out the mix by trial. We have always encouraged him in such cases to bring a sample of the aggregate used back to the laboratory and analyze it thus comparing his judgment against actual design. This coupled with the fact that we insist on all our field men being proficient in laboratory work enables them to use the trial method to better advantage. Also a field method for checking the bulking of the aggregate which we developed for these small jobs is a big help. To make this bulking test we fill a

reasonably tight 1 cu. ft. box with the sand or ready-mixed gravel as delivered; then turn a hose, not too forcibly, into it and allow it to run for about 5 minutes. Figuring the loss of volume as a percentage of the final volume checks the percentage of bulking very closely for the sand samples and fairly close on the ready-mixed gravel samples. This of course also serves as an indication of a change in water content of the aggregate.

FIG. 21. A BUS WALL AS TYPICAL OF SMALL JOB WORK.

The only measuring device we do insist upon is a good water tank and a 1 cu. ft. box. The aggregate on such jobs is usually a ready-mixed gravel and the methods of measuring it into the mixer will be described later.

We frequently have an opportunity to check the accuracy of our measurements in the finished work. Two such instances are as follows: On the Beacon Heating Plant job we poured the walls of two water storage tanks of the same size, requiring 86 cu. yd. of concrete in each tank. The pours were made 3 days apart and there were precisely 120 batches in each pour. The material was measured into a skip fitted

with an adjustable steel gate as shown in Fig. 18. The other example was on the Cortland Substation job, where we poured two bus walls of the same dimensions. These two pours were made on different days and required 68 batches (35½ cu. yd. concrete) each to complete them. This was measured in the same way as the Beacon job except that the gate used was merely a wooden partition placed in position by the laborers before dumping material into the skip and removed by them before raising the skip, as illustrated in Fig. 19.

The purpose of these examples is to show that controlling the measurement of the material on the smaller jobs does not necessarily mean expen-

FIG. 22. HEAVY CONCRETE MAT OVERHANGING FOREBAY AT ARGO HYDROPOWER PLANT.

sive equipment, but can be obtained when the inspector consistently checks up the men loading the skip to impress on them the necessity for careful measuring.

It is much easier from the standpoint of inspection to handle a concrete job where separated materials are being used than when aggregate is delivered ready mixed, and we hope that the concrete material situation will develop to the point where separated aggregates will be the rule instead of the exception. As this paper is being written the ready-mixed gravel is causing us so much trouble that we have decided to switch to separated aggregates on the substation jobs as well as the power plant jobs. A uniform and properly graded ready mixed material is getting steadily more difficult to get.

For the purpose of comparison we insert here a few photographs of the smaller jobs under control.

Fig. 20 illustrates a typical small transformer mat job.

Fig. 21 shows a bus wall in the Hazel Park Substation, just after the forms have been removed. These bus walls are difficult to pour because of the many inserts in them and the fact that puddling is prohibited lest it disturb these inserts. It is necessary to use an aggregate of ⅜-in. maximum size together with a slump of about 8-in.

Fig. 22 illustrates a heavy mat overhanging the forebay at our Argo Hydropower Plant. This mat is partially submerged at normal water level. Note the entire absence of honeycomb in the bottom of the mat.

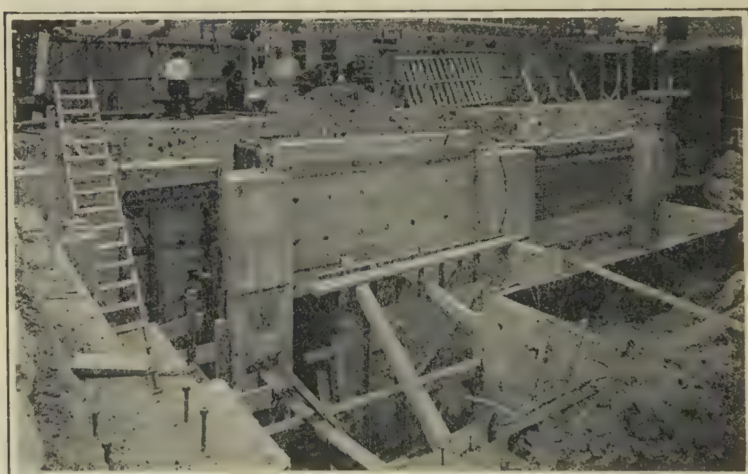


FIG. 23. TYPICAL FOUNDATION WORK ON SUBSTATION JOB.

Fig. 23 shows the class of foundation work encountered in our substation jobs although the heavy footings cannot be seen in this picture because the back-filling is completed.

Fig. 24 shows another type of transformer mat job.

These jobs were all controlled without the use of any special equipment and the concrete obtained was not only economical but of very high quality.

ECONOMY

Details have already been given to show that there is real economy in store for the engineer who takes the trouble to properly control the quality of his concrete.

To further illustrate this point we show Table No. 11, a summary of our estimated savings in the 3½ years we have had control methods in effect. These figures we believe are conservative. In the first place we

were pioneers in the work in this district and so were inclined to be very conservative at the start. The year 1925 shows only 8500 cu. yd. poured under control methods with a saving of 2150 bags of cement or only ¼ bag per yd. while in 1926 and 1927 the saving was better than 1 bag per yard.

Another limit was put on the economy when we established the 5-bag minimum on reinforced work in 1927. At that time we considered it the safest practice but in the light of more recent data on the durability

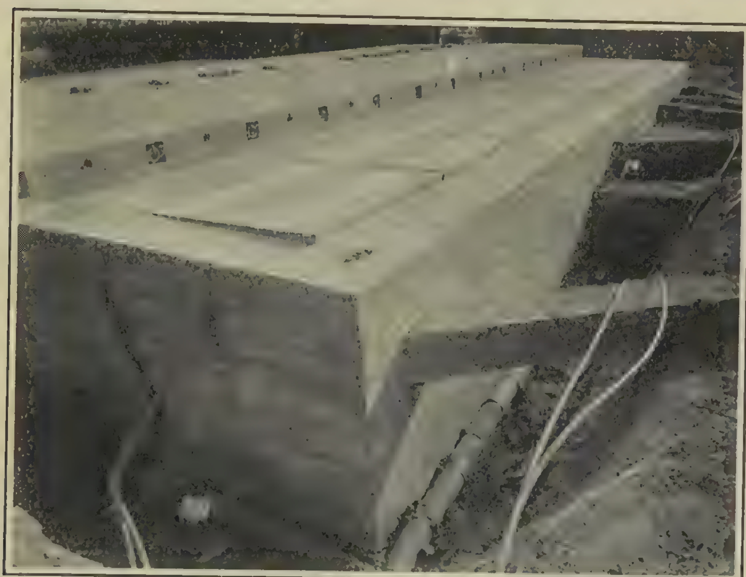


FIG. 24. A HEAVY TRANSFORMER MAT.

of the leaner concretes, given in detail further on, we believe that the minimum could safely be reduced to 4½ bags per cu. yd. if the job is under strict inspection and supervision. This is further borne out by the results thus far obtained in a series of permeability tests which we now have under way. The method of test is illustrated in Figs. 25 and 26. After the 7 and 28-day curing periods the pipes are filled with water and corked at the top to prevent evaporation. The loss in head is then recorded every day for a week. While the total head of water is only 50 in., we do find that when the other factors are kept constant the loss of head does not show much variation whether the cement content is 4½ bags per yd. or more, holding to a loss of from 1 to 2 inches. Below the 4½ bag limit the loss is greater and more erratic. We are endeavoring to make the test complete by varying the consistency, the sand ratios, the types of aggregates, together with the different standard and special

cements and admixtures. This, however, will not be completed for sometime.

Another point to be taken into consideration in considering this table of savings is that during rush jobs the superintendent of construction often requires a high-early-strength concrete in order that he may get his gangs back on it quickly. This we obtain by an extra bag of cement per cu. yd. and extra mixing, plus careful protection and curing. On the Delray job alone some 1500 cu. yd. were poured in this way.

DURABILITY

As mentioned in the remarks on economy, we established a dead line on minimum cement content until we could get some definite data on the durability of lean concretes.

In December, 1925, we started an experiment along this line by putting in three sections of road slab 10 x 10 ft. at our Delray Plant in a location that would subject them to the traffic of the trucks hauling away the cinders. These three slabs were 7 in. thick, reinforced with ⅜-in. bars on 12-in. centers both ways. After pouring they were protected by hay, tarpaulins and salamanders for three weeks, but received no curing until finally exposed to the weather.

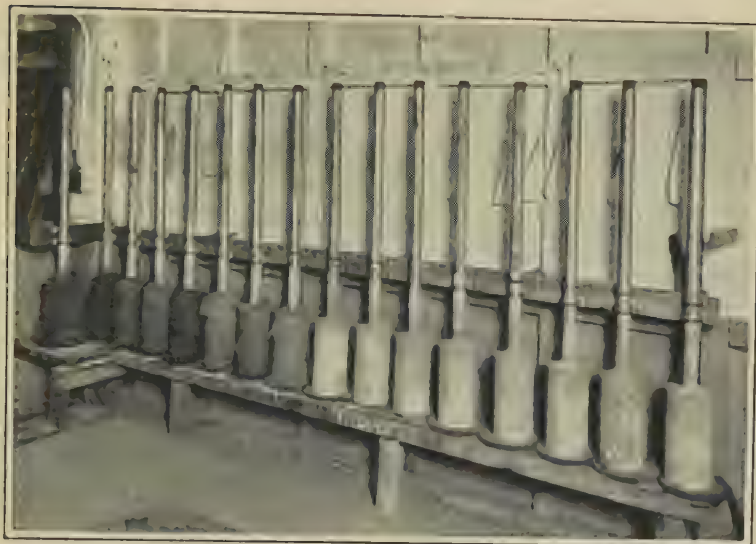
At the same time three slabs 3 x 3 ft. and 3 in. thick, reinforced by ¼-in. bars on 6-in. centers both ways were made from the same three mixes, and after curing for 28 days were hung in the river at the end of the dock where they would get the full force of the current. They were completely submerged and fastened only at the top which subjected them to considerable knocking around by the rough water. Table No. 12 gives the data on the mixes, the cement contents and the compressive strengths.

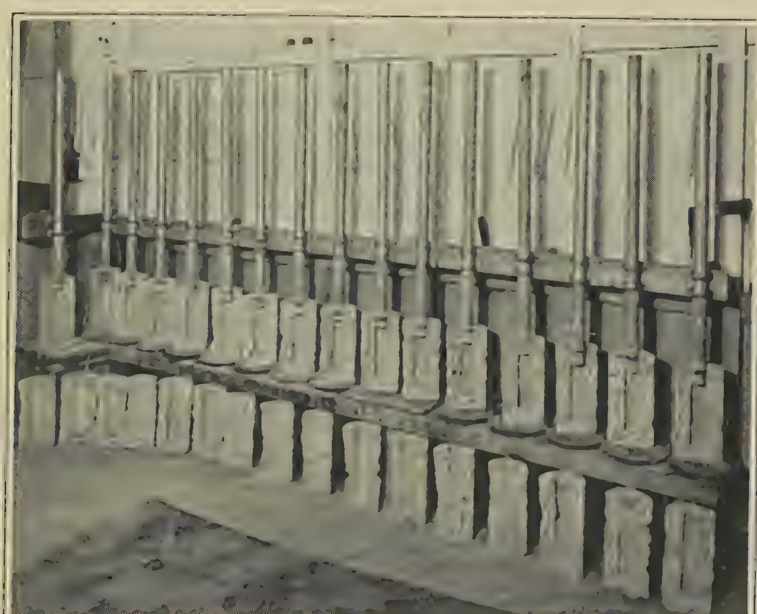
We have just had the road slabs dug up to make way for new work and have made a careful inspection of the concrete. To complete the story we have also removed the slabs from the river. Fig. 27 shows the slabs as they came out of the river. It will be noticed that they show the results of the rough knocking about. Fig. 28 shows the same slabs after vigorously brushing a portion of each with a wire brush. The concrete surface does not show any deterioration whatever on any of the slabs. Fig. 29 shows large specimens taken from the road slabs, with some smaller specimens broken from the corresponding slabs taken from the river. Fig. 30 is the same as Fig. 29 except that a portion of the surface of each specimen was wetted just before the picture was taken.

On breaking up the river slabs we found that the reinforcing was in perfect shape except where the edges or corners of the concrete had been broken away exposing it to the action of the water. The road slabs all developed a few cracks but they did not open up and at no time during the three years did we put any asphalt in them or doctor them up in any way.

The general conclusions are:

(1) That specimens No. 1 are a high-grade concrete, showing a mini-





FIGS. 25 AND 26. VIEWS OF SETUP FOR ABSORPTION TEST.

Pipes are filled with water, corked to prevent evaporation, and the loss of head measured daily.

mum of surface wear, reasonably dense, and should be satisfactory for most grades of reinforced concrete work.

(2) Specimens No. 2 are as good a grade of concrete as the average but show a whiter color than No. 1 and in our opinion are not good enough for reinforced concrete work.

(3) Specimens No. 3 show up somewhat better than No. 2.

CURING

Too much stress cannot be laid on the necessity for proper curing of concrete. So much data has been published to show conclusively that proper curing will increase the strength of concrete that we need hardly reproduce here the results of some actual tests we have made. It is sufficient to say that we have proved that properly cured concrete has shown 50 per cent greater strength at 28 days than concrete that had no curing whatever.

All our slab work is cured for 2 weeks with damp sawdust, and longer if the job conditions permit. Walls and columns will always be difficult to cure properly but we believe it helps to leave the forms on as long as possible and to spray the walls after removal of forms. On our larger jobs this is done for at least a week. Foundations will as a rule draw enough moisture from the surrounding ground.

Fig. 31 shows the relation between job-cured cylinders and those cured in damp sand for 28 days. All of the concrete included in this field test was designed 2500 lb. per sq. in, using the formula $S = \frac{14000}{8^x}$. The job cured cylinders were broken as soon as the curing was discontinued, and the standard cured cylinders were made in direct comparison, *i.e.*, on each day's run enough cylinders were made to allow for curing some on the job and others in the standard way.

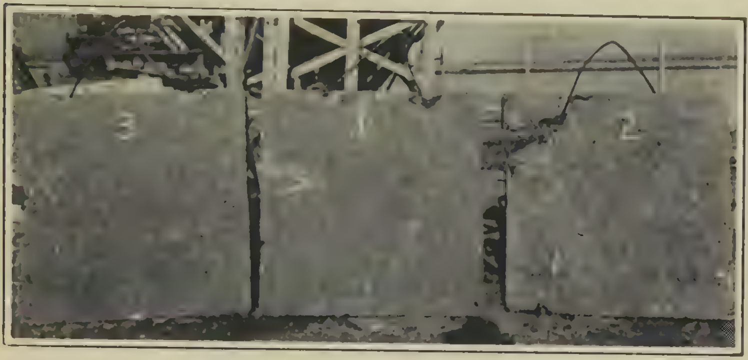
It is interesting to note that where the job curing has been continued the full 28 days the results are practically the same as the 28-day damp-sand curing. However, with the exception of foundation work below ground level, the concrete is rarely cured that long and in most cases 14 days is the limit. The average of the 14-day breaks on this curve is about 2400 lb. which it is safe to assume would have been at least 2700 lb. per sq. in. if the cylinders had been held till 28 days old without any further curing. This is not so far above the actual design and strengthens us in our opinion that it is better practice to use the more conservative curve $S = \frac{14000}{8^x}$.

WINTER METHODS

This paper would not be complete without a few comments on winter concreting methods. We have not found it desirable to use any anti-freeze compounds, so called, but in special cases where the work is exposed to extreme conditions we use calcium chloride to hasten the set and thus

shorten the time necessary for protection against frost. The length of this protection period in such cases is three days at least or until the concrete has attained more than one-quarter of its designed strength. This is checked by job cylinders broken before the protection is removed.

We require that the concrete poured in cold weather be held at a temperature of at least 60 deg. F. and 70 deg. F. if possible, which is



FIGS. 27 AND 28. THREE-INCH SLABS WHICH HAVE BEEN SUSPENDED IN RIVER THREE YEARS AS DURABILITY TEST.

Fig. 27, above, shows the slabs as they come from the river while in Fig. 28 portions of them have been brushed with a wire brush revealing no deterioration of any kind.

obtained, of course, by heating the aggregates and water. It is desirable on a water control job that the aggregates should not be heated too much, because the hot stones evaporate the water the minute they come together and it is difficult to maintain the desired water-cement ratio.

Any efficient method of protection the superintendent cares to use is satisfactory provided a temperature of from 50 to 70 deg. F. is maintained over all the exposed work; and the superintendents are required to have good thermometers on all jobs to keep a close check on this. The methods

adopted have been various and some interesting information has been obtained.

On the Pulford, Garfield and Hart Substation jobs we did some experimenting with hay and straw protection and found that hay is much better than straw, and that either one is more efficient if protected by a tight covering of tarpaulins or tar paper (preferably the former) well

FIGS. 29 AND 30. LARGE SPECIMENS OF 7-IN. ROAD SLABS IN SERVICE THREE YEARS COMPARED WITH SMALLER SPECIMENS OF SLABS IMMERSED IN RIVER THREE YEARS.

Fig. 29, above, and Fig. 30 are same except that in Fig. 30 a portion of each specimen was wetted just before picture was taken.

fastened down. Thermometers placed on the concrete under such protection have shown surprising results.

On the Hart job the roof slab was poured December 22, 1927, when the outside temperatures ranged from 25 to 35 deg. The concrete was about 60 deg. when placed and was protected by a layer of tar paper covered with from 4 to 6 in. of hay and tarpaulins; also salamanders on the floor 25 ft. below the roof. The next day the thermometers showed an average of 60 deg. on the top of the concrete.

On the Cortland roof, poured the same month with outside temperatures of from 18 to 28 deg. and high winds; with the salamanders only 8 ft. below roof and the hay 8 to 10 in. thick with no covering other than a wind-break at side of roof; the temperatures on the concrete averaged 40 deg. next day. Had the hay been covered with tarpaulins the results would undoubtedly have been better.

On the Pulford and Garfield jobs the pier footings were protected with a layer of tar paper and 12 to 18 in. of hay covered with tarpaulin tightly fastened down. These footings were poured in December, January and February, 1927, under outside temperatures of from 8 to 30 deg.

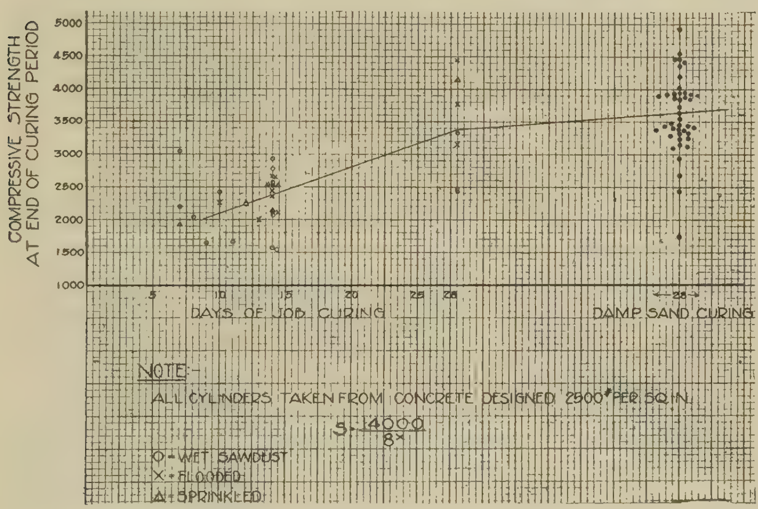


FIG. 31. RELATION BETWEEN JOB CURING AND STANDARD DAMP-SAND CURING.

The thermometers on the concrete registered from 50 deg. on the coldest days to as high as 75 deg. on the warmer days.

On the Delray Power House No. 3 foundations the concrete was protected from frost, and at the same time cured by live steam under tarpaulins. Wherever steam is available this method is most decidedly the best. As might be expected we found that it gave us a very high early strength. Table No. 13 gives the details of job cylinders made at the forms and subjected to the exact same conditions as the job concrete, as compared with others of the same batch cured in the standard way. Particular attention is called to the cylinders that were given the short-time steam curing and were then exposed to the weather with heavy frosts during the nights and milder temperatures during the days. It will be noted that the breaks are very satisfactory if the steam curing was continued for at least 24 hr., but that this time should be the absolute

minimum and 36 hr. would be advisable. It is interesting also to study the results on cylinders 3833 and 3835 in comparison with cylinders 3868 and 3869. In the first case one was cured in steam for 26 hr. and the other in damp sand, both protected the balance of the time, and the steam-cured specimen broke 25 per cent stronger than the other at age of 2 days. In the second case the steam curing was stopped at 13 hr. and the specimen, after exposure to the weather for one day, broke at about 40 per cent of the sand-cured protected cylinder. No doubt the large stones in this cylinder helped to cause such a low break but the appearance of the concrete indicated that the main reason was insufficient steam curing and protection.

The work on which these tests were made is mostly below ground level and consequently winds were not troublesome. Where the work is up in the air it is to be expected that slightly longer steam curing would be required to get these results unless the tarpaulins were exceptionally well placed.

MAXIMUM SIZE OF AGGREGATE

In 1926 we carried out a test to determine what effect, if any, the maximum size of aggregate had on the compressive strength of concrete when placed in small sections.

Seventy-two 6 x 12-in. cylinders were made using an aggregate of 2¼-in. maximum size. They were made in groups of 4 cylinders each, in 3 different slumps, 1, 4 and 7 in. and 6 different mixes ranging from a 1:3.5 real mix to a 1:7.0, for each slump.

Another series of 72 cylinders was made in exactly the same manner except that the maximum size of the aggregate used was 1½ in.

Fig. 32 shows the water-cement strength ratio curve for the 2¼-in. maximum size aggregate and Fig. 33 for the 1½-in. maximum size aggregate. It will be noticed that the aggregate of 1½ in. maximum size not only gives us a curve of higher strengths, $S = \frac{14000}{7.5^x}$, for all mixes and slumps, but it also gives less variation. The strength curve for the aggregate of 2¼ in. maximum size conforms to the equation $S = \frac{14000}{8^x}$ and the points do not adhere so closely to the curve.

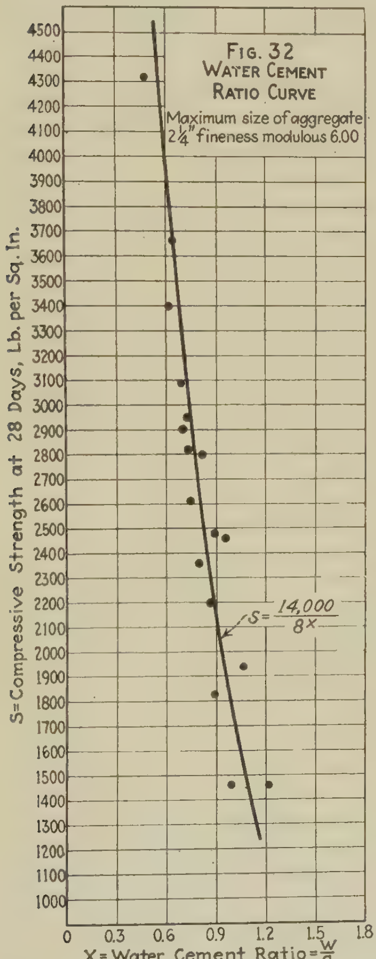
From this data we have adopted the standard practice of limiting the maximum size of aggregate to be used to ¼ the smallest dimension of the form in which it is to be placed.

SULPHUR WATER TESTS

In January, 1927, when we first encountered the presence of sulphur water in the caissons at Delray we went to considerable trouble to find out to what extent it affected concrete and what was the best means of combating it. Our investigations were extensive and data were collected from all parts of the country. Unfortunately, however, the information

was very conflicting in everything except the fact that it was something to worry about.

As we expect to have further extensions at Delray and possibly other



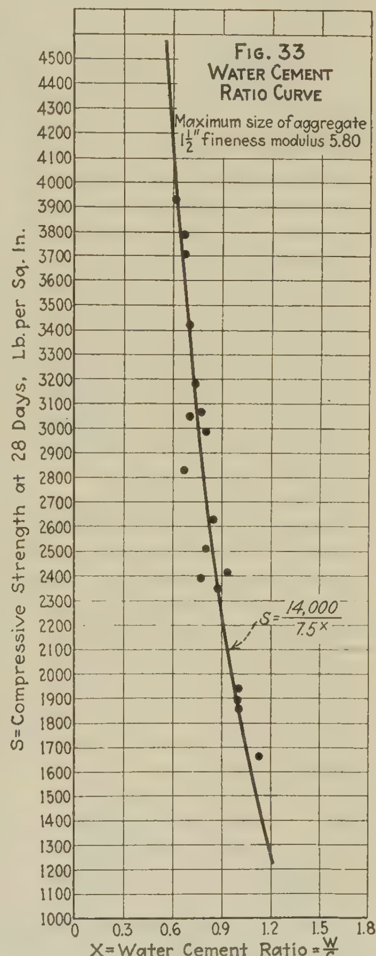


FIG. 32. WATER-CEMENT-STRENGTH CURVE FOR CYLINDERS CONTAINING 2¼-IN. MAXIMUM SIZE AGGREGATE.

FIG. 33. WATER-CEMENT-STRENGTH CURVE FOR CYLINDERS CONTAINING 1½-IN. MAXIMUM SIZE AGGREGATE.

jobs in the district where definite knowledge on this subject will be of value, we have started a test which should give us some definite answer. The method of test is as follows:

We drilled a cased hole and lined it with wood from ground level down into the rock until we struck a seam bearing sulphur water, which is under sufficient pressure to force it above the ground level. Alongside this well we built a tank and led a small stream of the sulphur water from the well into the tank. An overflow at the other end of the tank keeps the sulphur water circulating. A similar tank was made through which fresh water is kept circulating.

In December, 1927, we made approximately 300 test cylinders from our job concrete that went into the caissons, some with 7 bags of cement per yd., and some with 5¾ bags; using "super-cement" and portland cement in each of the mixes. The concrete that went into these cylinders was actually taken from the mixer during the pouring of one or more of the caissons. These cylinders are being cured in three different ways: one series in the sulphur water, another in the fresh water and a third in damp sand.

In June, 1928, some 300 more cylinders were made in the laboratory including specially rich mixes, as well as some admixtures, etc., and these also are being cured in the same manner.

Except for the 28-day tests we do not intend to break any more of these cylinders until about the two-year period and maybe not then if they show no signs of deterioration. Later on, however, we look for some valuable results.

COMPRESSIVE STRENGTH

The concrete design engineer is more and more coming to consider the compressive strength of concrete as a basis for his design and the field engineer is striving to obtain the desired compressive strength with uniformity and economy. He is forced to depend upon the accuracy of the breaking machine that he may buy or the commercial laboratory that he may employ. Table No. 14 shows the direct comparison of compressive strengths obtained on 5 machines in this district compared in each case with the machine in our own laboratory. For each comparison 12 cylinders were made from one batch of concrete. After curing for 7 days in damp sand, the 1st, 3rd, 5th, 7th, 9th and 11th cylinders were broken on our own machine, while at precisely the same time the 2d, 4th, 6th, 8th, 10th and 12th cylinders were broken on the other machine. The different series are not comparable as different materials and water-cement ratios were used.

While 3 of the machines checked well within 10 per cent the other 2 showed a variation of 17½ per cent and 57 per cent which is not so good. It would seem that some uniform method of calibration should be adopted and that all machines should be checked frequently.

The writer would also like to have further data as to the actual compressive strength of concrete in large masses in comparison with the 6 x 12 in. cylinder. This size of cylinder is the standard and it would not be practical to use one much larger. However, there seems to be ample data to substantiate the fact that cylinders of larger size show lower

compressive strengths. Our own experiments along this line have not been extensive but we have made a few direct comparisons between 8 x 16 in. and 6 x 12 in. cylinders and have found that the larger specimens did break at lower unit strengths.

Job Cylinders.—On all jobs of any extent we use iron moulds for making the test cylinders. They are made in our own shop from 12-in. sections of 6-in. di. pipe cut longitudinally into two halves, and are held together with an iron band, brought up tight with a bolt and wing nut. They are set up on machined plates and care is always taken to have a

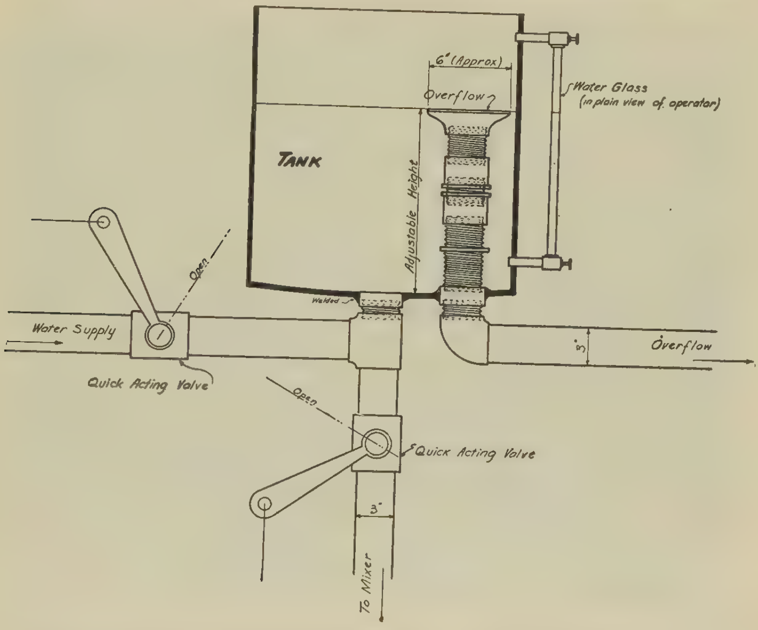


FIG. 34. SPECIAL WATER MEASURING DEVICE DEVELOPED BY DETROIT EDISON COMPANY.

good, level, plank bench made on which to set them up. On the smaller jobs waxed paper moulds are sometimes used.

We endeavor to make the cylinders in the field ready for breaking in the machine, without the necessity of capping them in the laboratory. The method adopted has been to fill the mold slightly above the top and after the cement has its initial set to level it off and trowel to a good surface. This probably gives a lower break than the laboratory capping but on the other hand the moulds are not absolutely water-tight and there is always a certain amount of leakage of water which of course would lower the water-cement ratio and tend to increase the strength of the concrete in the cylinders.

If the maximum size of the aggregate being used is larger than 1½ in. there is no attempt made to take out the larger stones but the sample is made from the concrete as it comes which of course is not according to standard practice. Many low breaks are reported by the laboratory as being due to an excess of this larger stone and this is probably one reason we get a greater variation in our breaks. However, we feel it is more desirable than picking out the larger stone when making the cylinders thereby getting a higher break of uncertain value. The bulk of our cylinders are cured in damp sand for 28 days but wherever the question

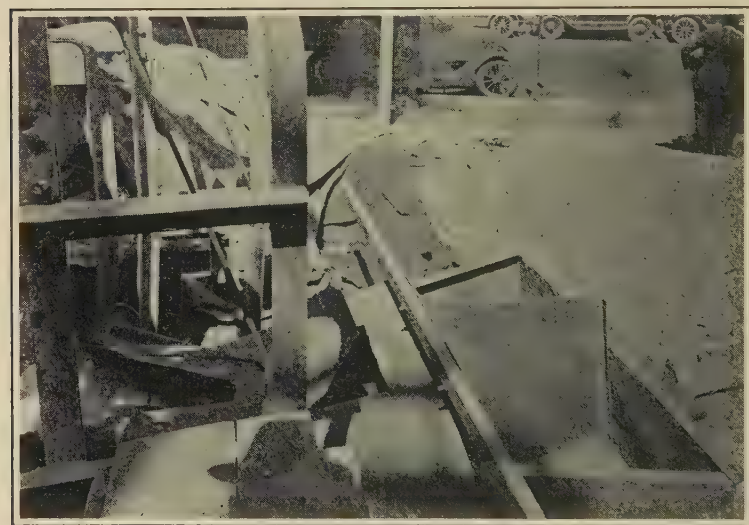


FIG. 35. METHOD OF DIVIDING MIXER SKIP FOR ACCURATELY MEASURING SEPARATED AGGREGATES.

Note hole in skip apron for charging cement in front. See also Figs. 18 and 19 for method used with ready-mixed materials.

of removing supports from slabs or discontinuing frost protection is likely to arise, cylinders are cured under job conditions and tested before permission is given to remove such supports or protection.

MEASURING DEVICES

The water tank as shown in Fig. 34 is a development of our own and has been our standard for the past 2 years. It has been found to be very efficient and quick acting which is an important factor. The 6-in. diameter saucer-shaped top on the overflow pipe was designed to speed up the overflow and avoid the danger of having the operator open the valve to the mixer before the excess of water has drained off.

It will be noticed that the sleeves and collars are held by a lock nut so that the vibrating of the mixer does not change the adjustment. If

these sleeves and collars are made of non-rusting metal it is easier to keep them in good repair.

The devices used on larger jobs for measuring the aggregates are volume batchers, weighing batchers or inundators. On the small jobs different methods of measuring the material into the skip are adopted the one most commonly used being the gates in the skips, shown in Figs. 18 and 19. These were used with a ready-mixed material. Fig. 35 shows how such a method was adapted to separated materials. The apron or gate in front is removable while the other partition is stationary.

FIG. 36. MEASURING HOPPER DISCHARGING THROUGH GATE INTO SKIP.

Note also the provision made for putting the cement in the front of the skip.

We also find that we can get excellent results by measuring the material in properly calibrated wheelbarrows and this method saves time when using separated materials. To get accuracy by this method, very close supervision and careful screeding is necessary.

Figure 36 illustrates a measuring hopper set up in front of the skip which was used with considerable success. The bottomless box which was the first method adopted on the Beacon Heating Plant job (Fig. 3) is still used occasionally but the other methods have been found quicker and less cumbersome.

The concrete equipment manufacturers have a large field for progress in perfecting measuring devices, particularly for use with their smaller-size mixers. There would seem to be great possibilities in the design of loading skips that will help the contractor on a small job to get the proper amount of material into each batch. Also many of the so-called automatic water tanks are not fool proof and in most cases depend on the operator entirely.

ADVANTAGES OF CONTROL

We have found in our three and a half years experience that controlling the quality of our concrete has many advantages and the enumeration of them would seem a logical conclusion to this paper.

(1) We do get a concrete of known minimum strength. The records accompanying this paper show several excessively low breaks but we have not eliminated them because we are telling the whole story. Practically all of these low breaks, however, were reported by the laboratory as being due to too much large aggregate, or other causes within the cylinder itself.

While strength is not the whole story, control methods if properly carried out will also control the workability, durability, impermeability and placing of the concrete.

(2) We find that it does give us a uniform concrete.

(3) We do not have any trouble with laitance.

(4) Enough data has been tabulated to show that there is a real saving in cement without skimping. As the methods of handling aggregates improve more economies will follow.

We believe also that it means economy in labor costs. In this connection we again wish to emphasize the fact that quality control does not mean simply analyzing the materials and designing the batch. It means a careful control over all the concreting operations beginning at the source of supply of the materials and ending only when the concrete is properly placed in all parts of the forms, without undue handling; and properly cured.

It is very noticeable on our jobs that the laborer is fresher at the end of a long run than was the case under the old heart-breaking methods of trying to make the concrete flow into forms out of reach.

(5) It creates a much greater familiarity with the concrete materials and with quantities and yields.

(6) It permits a more intelligent application of the best methods for obtaining high-early-strength concretes.

(7) It permits a more accurate cost study per cubic yard of concrete before the start of the job and provides a valuable record of the job.

(8) Last but by no means least, it certainly eliminates about 95 per cent of the worry for the engineer and superintendent.

TABLE I.—VARIATION IN THE MATERIAL COST PER CU. YD. OF CONCRETE DUE TO ALL CAUSES

Date	Sample	Location	Fineness Modulus	Slump, inches	Bulk, per cent	Cement, bags per cu. yd. of Concrete	Strength at 28 Days, lb. per sq. in.		Material Cost per cu. yd. of Concrete	Remarks
							Design	Test		
5-27-25	314	Madison	4.85	6-7	13	9.31	2500	3160	8.24	
7-1-25	357	"	5.00	3-4	12	6.92	2500	3630	6.73	
6-23-25	349	"	5.00	8-10	10	12.85	2300	3390	10.36	Bus Wall Cement, 62½c. per bag Gravel, $2.14 per yd.
9-17-25	567	"	5 30	3-4	14	6.00	2300	....	6.19	
6-12-25	340	"	5.04	6-7	10	8.11	2000	3410	7.42	
6-20-25	347	"	5.02	3-4	10	5.63	2000	2910	5.87	
4-11-25	101	Beacon	5.65	3-4	20	9.80	3500	3990	8.68	
6-6-25	336B	"	6.36	½-1	10	6.75	3500	2830	6.57	
8-19-25	552	"	5.00	6-7	13	9.31	2500	3210	8.24	Cement, 62½c. per bag Gravel, 2.14 per yd.
5-29-25	323	"	5.80	½-1	20	4.90	2500	2410	5.61	
7-18-25	370	"	5.10	3-4	10	5.51	2000	2600	5.79	
5-13-25	193	"	5.70	½-1	20	4.09	2000	2695	5.11	
10-8-25	752	Marysville	5.92	6-7	18.3	7.70	2500	2700	5.37	
9-19-25	443	"	6.21	6-7	18	6.59	2500	1590	4.82	
8-3-25	432	"	4.82	6-7	15.6	8.18	2300	3050	5.56	Cement, 49c. per bag Gravel, $1.35 per yd.
9-12-25	441	"	6.21	3-4	16.5	5.00	2300	2960	4.03	
5-13-25	147	"	4.81	6-7	17	7.71	2000	3130	5.36	
11-21-25	778	"	5.99	3-4	13	4.65	2000	1980	3.81	
8-25-25	520	Maple Warehouse	5.04	6-7	13	9.31	2500	3010	8.31	
10-29-25	803	Fort St. Coal Station	4.80	3-4	15	7.30	2500	2560	7.05	Cement, 65c. per bag Gravel, $2.00 per yd.
9-28-25	544	Elmdale Substation	5.31	6-7	13	7.94	2300	1770	7.42	
8-24-25	528	Maple Warehouse	5.73	3-4	13.5	5.11	2300	2850	5.59	
5-15-25	162	West Warren	4.86	6-7	26.6	7.60	2000	2850	7.46	Cement, 65c. per bag Gravel, $2.00 per yd.
12-4-25	812	Navarre	5.77	3-4	12	4.65	2000	1130	5.26	
7-27-25	492	Delray	5 50	3-4	17.9	7 71	2300	2510	7.17	
10-19-25	709	"	5.80	3-4	15	5.40	2300	2910	5.68	Cement, 62½c. per bag Gravel, $2.00 per yd.
9-17-25	505	"	5 01	6-7	13 2	7.29	2000	3400	6.83	
10-13-25	705	"	5 78	3-4	17.8	4.91	2000	2270	5.41	

TABLE 2.—VARIATION IN MATERIAL COST PER CU. YD. OF CONCRETE DUE TO CHANGE IN FINENESS MODULUS.

Date	Sample	Location	Fineness Modulus	Slump, inches	Bulk, per cent	Cement, bags per cu. yd. of Concrete	Strength at 28 Days, lb. per sq. in.		Material Cost per cu. yd. of Concrete	Remarks
							Design	Test		
5-21-25	310	Madison	4.6	6-7	*13	6.14	1500	1705	6.25	
6-19-25	346	"	4.7	6-7	13	6.00	1500	2930	6.17	
5-28-25	316	"	4.84	6-7	13	5.74	1500	3090	6.01	Cement, 62½c. per bag Gravel, $2.14 per yd.
5-18-25	302	"	4.7	3-4	*20	5.87	2000	2310	6.24	
5-9-25	189	"	5.0	3-4	19.3	5.63	2000	2410	6.07	
5-29-25	321	"	5.2	3-4	*20	5.40	2000	2510	5.94	
8-19-25	552	Beacon	5.01	6-7	*10	9.31	2500	3210	8.17	
7-18-25	371	"	5.1	6-7	10	9.00	2500	2750	7.98	
7-25-25	378	"	5.45	6-7	10	8.44	2500	3440	7.61	Cement, 62½c. per bag Gravel, $2.14 per yd.
7-8-25	364	"	5.1	3-4	*10	5.6	2000	3420	5.86	
7-3-25	361	"	5.5	3-4	10	5.0	2000	1870	5.48	
6-19-25	344	"	6.06	3-4	10	4.5	2000	2170	5.16	
8-4-25	433	Marysville	4.82	6-7	*12	6.43	2000	3050	4.66	
10-24-25	765	"	5.92	6-7	11	6.14	2000	2540	4.50	
11-7-25	774	"	6.00	6-7	13	5.87	2000	2170	4.41	
8-3-25	431	"	4.82	3-4	18	6.75	2300	3390	4.90	Cement, 49c. per bag Gravel, $1.35 per yd.
9-12-25	442	"	6.21	3-4	16.5	5.0	2300	2590	4.03	
10-5-25	751	"	5.92	6-7	18.3	7.71	2500	2160	5.37	
9-19-25	443	"	6.21	6-7	18	6.59	2500	1590	4.82	
12-5-25	813	Navarre	4.42	3-4	12	5.87	2000	1530	6.06	
10-5-25	545	Connor's Creek	5.00	3-4	12	5.60	2000	3101	5.89	Cement, 65c. per bag Gravel, $2.00 per yd.
12-4-25	812	Navarre	5.77	3-4	12	4.65	2000	1130	5.26	
10-19-25	716	Delray	5.80	3-4	15	5.40	2300	2640	5.68	Cement, 62½c. per bag Gravel, $2.00 per yd.
12-16-25	747	"	5.77	3-4	15	5.51	2300	2470	5.74	
9-17-25	505	Delray	5.01	6-7	13.2	7.29	2000	3400	6.82	Cement, 62½c. per bag Gravel, $2.00 per yd.
10-2-25	701	"	5.7	6-7	*13.2	6.28	2000	2930	6.20	

* Items marked with an asterisk have been adjusted to afford a direct comparison of fineness modulus cost.

TABLE 3.—VARIATION IN MATERIAL COST PER CU. YD. OF CONCRETE DUE TO CHANGE IN SLUMP

Date	Sample	Location	Fineness Modulus	Slump, inches	Bulk, per cent	Cement, bags per cu. yd. of Concrete	Strength at 28 Days, lb. per sq. in.		Material Cost per cu. yd. of Concrete	Remarks
							Design	Test		
5-27-25	315	Madison	4.85	6-7	13	9.31	2500	2600	8.24	Bus Wall Cement, 62½c. per bag Gravel, $2.14 per yd.
6-4-25	331	"	4.88	3-4	*13	7.10	2500	3840	6.85	
6-23-25	349	"	5.0	8-10	10	12.85	2300	3390	10.36	
6-8-25	337	"	5.0	6-7	9.5	8.43	2300	2540	7.60	
6-12-25	341	"	5.04	6-7	10	8.11	2000	2837	7.42	
8-20-25	556	"	5.0	3-4	10	5.63	2000	2610	5.87	
7-18-25	371	Beacon	5.1	6-7	10	9.00	2500	2750	7.98	Cement, 62½c. per bag Gravel, $2.14 per yd.
7-22-25	376	"	5.1	3-4	10	6.75	2500	2590	6.57	
9-10-25	564	"	4.8	6-7	10	8.71	2300	2770	7.77	
9-9-25	562	"	4.8	3-4	10	6.58	2300	2720	6.44	
7-2-25	359	"	5.0	3-4	*10	5.62	2000	3490	5.86	
8-18-25	551	"	5.01	½-1	10	4.74	2000	2830	5.27	
11-18-25	776	Marysville	6.0	6-7	13	6.75	2300	2500	4.84	Cement, 49c. per bag Gravel, $1.35 per yd.
9-12-25	442	"	*6.0	3-4	*13	5.2	2300	2590	4.07	
8-3-25	432	"	4.82	6-7	15.6	8.18	2300	3050	5.56	
8-3-25	431	"	4.82	3-4	*15.6	6.75	2300	3390	4.86	
5-12-25	146	"	4.81	6-7	17	7.71	2000	3135	5.36	
7-23-25	428	"	4.82	3-4	*17	5.87	2000	2710	4.46	
8-20-25	529	Maple Warehouse	5.73	6-7	13.5	6.4	2000	2300	6.42	Cement, 65c. per bag Gravel, $2.00 per yd. Test cylinder poorly made
12-4-25	812	Navarre	5.77	3-4	12	4.65	2000	1130	5.26	
10-2-25	701	Delray	5.7	6-7	17.8	6.28	2000	2930	6.29	Cement, 62½c. per bag Gravel, $2.00 per yd.
9-23-25	508	"	5.7	3-4	17.8	4.91	2000	3520	5.41	
8-2-25	495	"	5.48	6-7	17.9	7.70	2300	3400	7.15	
12-9-25	744	"	*5.48	3-4	*17.9	5.74	2300	1290	5.95	

* Items marked with an asterisk have been adjusted to afford a direct comparison of slump costs.

TABLE 4.—VARIATION IN MATERIAL COST PER CU. YD. OF CONCRETE DUE TO CHANGE IN BULKING.

Date	Sample	Location	Fineness Modulus	Slump, inches	Bulk, per cent	Cement, bags per cu. yd. of Concrete	Strength at 28 Days, lb. per sq. in.		Material Cost per cu. yd. of Concrete	Remarks
							Design	Test		
5-26-25	311	Madison	5.0	3-4	26.6	5.63	2000	3310	6.22	Cement, 62½c. per bag Gravel, $2.14 per yd.
5-9-25	188	"	5.0	3-4	19.3	5.63	2000	3250	6.07	
6-30-25	352	"	5.0	3-4	12.0	5.63	2000	3490	5.92	
8-20-25	556	"	5.0	3-4	10.0	5.63	2000	2610	5.87	
7-8-25	363	Beacon	5.09	3-4	13.8	5.62	2000	3210	5.94	Cement, 62½c. per bag Gravel, $2.14 per yd.
7-17-25	368	"	5.11	3-4	10.0	5.51	2000	2440	5.79	
11-3-25	598	"	*5.5	6-7	20.0	6.59	2000	2960	6.69	
7-3-25	360	"	5.5	6-7	10.0	6.58	2000	3300	6.44	
10-5-25	751	Marysville	5.92	6-7	18.3	7.71	2500	2160	5.37	Cement, 49c. per bag Gravel, $2.14 per yd.
10-16-25	758	"	5.92	6-7	11.2	6.00	2500	2480	4.44	
10-2-25	450	"	5.92	6-7	18.5	7.10	2300	2340	5.07	
10-16-25	757	"	5.92	6-7	11.2	7 10	2300	3100	4.98	
10-5-25	545	Connor's Creek	5.0	3-4	12	5.6	2000	3101	6.05	Cement, 65c. per bag Gravel, $2.14 per yd.
6-13-25	174	Mandalay	*5.0	3-4	20.1	5 62	2000	2780	6.22	
5-19-25	155	Charlotte	*5 0	3-4	24.2	5 51	2000	2480	6.24	
8-24-25	526	Maple Warehouse	*5.77	3-4	13.5	5 51	2300	2960	5.72	Cement, 62½c. per bag Gravel, $2.00 per yd.
11-10-25	723	Delray	5.77	3-4	15	5 51	2300	2100	5.74	

NOTE.—It is assumed that bulking as shown above actually affected the cost of the gravel, because on most of the jobs no stock was carried and gravel was used the day it was delivered.

Where gravel is stocked, the bulking at the time of delivery would be the only one to affect the cost.

* Items marked with an asterisk have been adjusted to afford a direct comparison of cost due to bulking.

TABLE 5.—COMPARISON OF MATERIAL COSTS PER CU. YD. OF CONCRETE, SEPARATED AGGREGATES OR RIVER GRAVEL.

Date	Sample	Location	Fineness Modulus	Slump, inches	Bulk, per cent	Cement, bags per cu. yd. of Concrete	Strength at 28 days, lb. per sq. in.		Material Cost per cu. yd. of Concrete	Remarks
							Design	Test		
SEPARATED AGGREGATES										
11-3-25	713	Delray	6.05	3-4	*	4.42	2000	2590	4.93	Cement, 62½c. per bag; Sand, $1.29 per yd.; Pebbles, $2.10 per yd.
11-6-25	718	"	6.05	3-4	*	4.42	2000	2240	4.93	
11-13-25	726	"	6.05	3-4	*	4.42	2000	1950	4.93	
11-23-25	731	"	6.05	3-4	*	4.42	2000	2480	4.95	
12-21-25	822	"	6 05	3-4	*	4.42	2000	3410	4 95	
RIVER GRAVEL										
10-13-25	705	Delray	5.78	3-4	17.8	4.91	2000	2270	5.41	Cement, 62½c. per bag; Gravel, $2.00 per cu. yd.
10-5-25	545	Connor's Creek	5.0	3-4	12	5.6	2000	3101	5.89	Cement 65c. per bag; Gravel, $2.00 per cu. yd.
5-7-25	153	Charlotte	5.107	3-4	24.2	5.51	2000	2745	6.12	Cement, 62½c. per bag; Gravel, $2.25 per cu. yd.
6-8-25	159	"	5.22	3-4	22.4	5.4	2000	2320	6.13	Cement, 62½c. per bag; Gravel, $2.25 per cu. yd.
5-18-25	302	Madison	4.7	3-4	33.3	5 87	2000	2310	6.52	Cement, 62½c. per bag; Gravel, $2.14 per cu. yd.
5-29-25	321	"	5.2	3-4	22	5.4	2000	2510	5.99	Cement, 62½c. per bag; Gravel, $2.14 per cu. yd.
6-5-25	332	"	4.87	3-4	10.1	5.74	2000	3170	5.94	Cement, 62½c. per bag; Gravel, $2.14 per cu. yd.
8-20-25	556	"	5.0	3-4	10	5.63	2000	2610	5.87	Cement, 62½c. per bag; Gravel, $2.14 per cu. yd.
6-19-25	344	Beacon	6 0	3-4	10	4.5	2000	2170	5.16	Cement, 62½c. per bag; Gravel, $2.14 per cu. yd.
7-10-25	366	"	5.3	3-4	12	5.29	2000	2630	5.12	Cement, 62½c. per bag; Gravel, $2.14 per cu. yd.
7-23-25	428	Marysville	4.82	3-4	13	5.87	2000	2710	4.41	Cement, 49c. per bag; Gravel, $1.35 per cu. yd.
11-21-25	778	"	5 99	3-4	13	4.65	2000	1930	3.81	Cement, 49c. per bag; Gravel, $1.35 per cu. yd.

* Inundator eliminated bulking, but for costs figured 20 per cent on sand, 0 per cent on stone.

TABLE 6.—VARIATION IN WEIGHT OF CEMENT DELIVERED IN BAGS, JULY TO OCTOBER, 1926

Brand of Cement	Number of Tests	Average Net Weight	Maximum Net Weight	Per Cent Variation Above Standard Weight	Minimum Net Weight	Per Cent Variation Below Standard Weight	Per Cent Over All Variation
A	47	95 lb. 4 oz.	99 lb. 4 oz.	5.6	85 lb. 13 oz.	8.7	14.3
B	88	94 lb. 5 oz.	100 lb. 8 oz.	6.9	86 lb. 8 oz.	8.0	14.9
C	113	96 lb. 12 oz.	100 lb. 14 oz.	7.3	91 lb. 8 oz.	2.7	9.9
D	5	97 lb. 8 oz.	99 lb. 3 oz.	5.5	96 lb. 4 oz.	...	5.5
E	4	99 lb. 0 oz.	101 lb. 5 oz.	7.7	95 lb. 4 oz.	...	7.7
F	6	97 lb. 0 oz.	98 lb. 15 oz.	5.2	95 lb. 7 oz.	...	5.2
G	30	94 lb. 8 oz.	99 lb. 0 oz.	5.3	90 lb. 0 oz.	4.2	9.5
Average	263	95 lb. 12 oz.					

TABLE 7.—SUMMARY OF 1926 CONCRETE WORK.

Job	Concrete, cu. yd.	Cement, bags per cu. yd. of Concrete	Designed Strength, lb. per sq. in.							
			2700		2300		2000		1800	
			Cylinders	Average Break	Cylinders	Average Break	Cylinders	Average Break	Cylinders	Average Break
Beacon	3,077.76	5.67	55	3190	11	3320	25	2820	10	2240
Charlotte	444.46	5.35	..	....	23	3370	43	2770	..	....
Connor's Creek	482.54	5.01	..	....	..	....	32	2950	..	....
Dearborn	70.30	4.79	..	....	..	....	1	2000	7	1925
Delray	1,029.33	4.69	..	....	23	2880	84	2594	..	....
Ecorse Warehouse	353.79	5.26	..	....	..	....	32	2410	..	....
McKinstry	878.17	5.60	..	....	..	....	36	2800	24	2520
Marysville	2,322.79	4.57	..	....	..	....	65	2800	..	....
Northeast	953.49	4.09	..	....	..	....	29	2430	14	2630
Redford	246.41	5.86	..	....	..	....	19	2350	..	....
Roseville	220.01	4.63	..	....	..	....	19	2340	..	....
Savannah	157.66	5.52	..	....	..	....	22	2900	..	....
Shelby	459.95	4.63	..	....	..	....	30	2430	..	....
Stoepel	308.95	5.26	..	....	..	....	31	2270	..	....
St. Louis	210.20	5.14	..	....	..	....	19	2870	7	2910
Totals	11,215.81	....	55	3190	57	3160	487	2640	62	2475

Grand average, cement per cu. yd. concrete, 5.16 bags.

TABLE 8.—ADMIXTURE TESTS.

Admixture	Cylinders Made	How Used	Compressive Strength, lb. per sq. in. (Average of 4 Cylinders) Designed $S=\frac{14000}{8^x}$	7 Days	28 Days	60 Days	4 Mos.	6 Mos.	Slump	$\frac{W}{C}$	Yield, per cent	Material Cost per cu. yd.
None	20	....	2500	2080	3070	4050	4710	4200	4½	0.93	100	4.87
None	20	....	2500	2495	3620	4395	4340	4470	4	0.91	100	4.90
A	20	Substituted for Cement............	2500	1590	3095	3435	4040	3960	4	1.07	102	5.04
A	20	Substituted for sand	2500	2110	3660	3490	4260	4380	4	0.99	99	5.36
A	20	Added..............	2500	2000	3240	3560	4005	4220	4	1.05	103	5.21
	20	Cement added equivalent value.......	2500	2620	4245	3550	5190*	5190*	4⅞	0.84	102	5.32
B	20	Substituted for cement............	2500	1775	3340	3240	4390	4555	4½	1.07	97	5.36
B	20	Substituted for sand.	2500	2145	3670	4030	4605	4675	3½	0.95	98	5.24
B	20	Added..............	2500	2050	3360	4010	4670	4980	3¼	1.00	102	5.11
	20	Cement added equivalent value.......	2500	2510	3905	4430	5150	5225*	4¼	0.86	102	5.09
C	20	5 per cent hydrated lime added.......	2500	1850	2720	3560	4115	...	4½	0.96	102	4.96
D	20	Added to gauging water............	2500	1800	2560	3230	4210	...	3½	0.89	99.5	6.72
E	20	Added to gauging water............	2500	1945	2840	3170	...	...	3¼	0.89	99.5	6.53
F	20	Special cement......	2500	1175	2180	2450	2930	...	3½	0.96	100	5.38
G	20	Special cement......	2500	2100	3390	3540	4320	...	4	0.93	99	6.26

NOTE.—All cylinders cured in damp sand for 60 days. Balance of time in air.
All batches:
Real mix 1:5.
Fineness modulus 5.70.
Some of the cylinders have not been broken.
* Cylinders marked with * did not break at capacity of machine.

TABLE 9.—SUMMARY OF CONCRETE DATA, DELRAY POWER HOUSE NO. 3

Where Placed	Concrete, cu. yd.	Cement, bags	Average Bags per cu. yd.	Concrete Designed $S=\frac{14000}{8^x}$ Compressive Strength, lb. per sq. in.	Concrete, minimum bags per cu. yd.	Number of Test Cylinders	Average Compressive Strength, lb. per sq. in.	Variation
Caissons, lower half...	5,354	39,357	7.35	3500	7.00	198	5110	3040–7650
Caissons, upper half...	3,287	19,549	5.94	2900	5.75	92	4170	2300–5300
Substructure..........	20,883	107,224	5.13	2500	5.00	286	3410	1730–4820
Superstructure........	3,289	18,133	5.51	2500	5.00	48	3510	2290–4740
Grand totals..........	32,813	184,263	5.61					

TABLE 10.—SUMMARY OF 1927 CONCRETE WORK.

Job	Concrete, cu. yd.	Cement, bags per cu. yd.	Designed Strength, lb. per sq. in. $S=\frac{14000}{8^x}$											
			4000		3500		3000		2500		2000		1500	
			Cylinders	Average Break	Cylinders	Average Break	Cylinders	Average Break	Cylinders	Average Break	Cylinders	Average Break	Cylinders	Average Break
Argo	298	5.53	...	....	...	....	5	3945	...	....	5	2870	..	....
Beacon	489	4.67	...	....	...	....	..	....	2	3360	19	2915	6	1735
Bloomfield	726	4.97	...	....	...	....	18	3720	...	....	37	2300	..	....
Cadillac mat	148	4.34	...	....	...	....	..	....	...	....	5	3550	..	....
Cortland	1,222	5.34	...	....	...	....	33	3300	...	....	23	2710	11	2370
Delray	10,083	5.00	...	....	...	....	14	3560	192	3127	23	2770	..	....
Delray Caissons	5,425 3,346	7.25 6.00	39	5420	153	5000	94	4150	...	....	3	3340	..	....
Farmer	2,406	5.57	...	....	...	....	..	....	39	3800	20	3140	6	2930
Fenkell Garage	261	4.84	...	....	...	....	..	....	...	....	19	3270	..	....
Garfield	1,238	4.75	...	....	...	....	..	....	11	2800	53	2700	15	1700
Hart	356	4.80	...	....	...	....	..	....	...	....	17	2900	3	2180
Hazel Park	284	4.85	...	....	...	....	..	....	...	....	27	3095	3	2640
Marysville	4,514	4.05	...	....	...	....	2	3830	...	....	96	2790		....
Moran	983	5.46	...	....	...	....	8	2905	14	2835	29	3120	13	2570
Mt. Olivet	350	4.24	...	....	...	....	..	....	2	2545	22	2550	..	....
Northeast	204	4.35	...	....	...	....	..	....	...	....	14	2380	..	....
Pingree	245	4.97	...	....	...	....	..	....	...	....	22	2780	..	....
Pulford	1,734	4.88	...	....	...	....	..	....	6	3550	52	3230	19	2480
Service Bldg	1,041	5.33	...	....	...	....	..	....	...	....	38	3030	6	2260
Superior	267	5.17	...	....	...	....	..	....	...	....	15	3060	..	....
Willis	522	4.91	...	....	...	....	..	....	...	....	34	2790	..	....
Totals	36,142	....	39	5420	153	5000	174	3830	280	3040	577	2865	82	2300
Cylinders that broke below design strength			0	....	3	2%	12	6.9%	11	5.2%	12	2%	7	8.5%
Cylinders that did not break at capacity of machine, 5200 lb. per sq. in.			13	33.6%	79	51.5%	3	1.7%	..	....	..	....	..	....

Grand average cement per cu. yd. concrete .... 5.33 bags
Grand average cement per cu. yd. concrete (exclusive of Delray Caissons) .... 4.87 bags

TABLE 11.—ESTIMATED SAVING IN CEMENT.

Year	1800 to 2500 lb.		Greater than 2500 lb.		Average Bags Cement per cu. yd.	Savings in Cement based on		Total Bags Saved	Average Cost per bbl.	Total Saving
	Concrete, cu. yd.	Cement, bags	Concrete, cu. yd.	Cement, bags		6 Bags per cu. yd. Concrete, 1:2:4 mix	7½ Bags per cu yd. Concrete, 1:1½:3 mix			
1925.... From May 1	6,379	36,835	......	......	5.77	0.23	....	1,467	$2.25	$825.75
	......	......	2,129	15,288	7.18	....	0.32	681	....	382.50
1926......	10,905	54,598	......	......	5.00	1.00	....	10,905	2.45	6,678.70
	......	......	1,611	9,376	5.82	....	1.68	2,706	....	1,658.65
1927.....	24,071	116,311	......	......	4.83	1.17	....	28,163	2.15	15,138.15
	......	......	10,400	68,956	6.63	....	0.87	9,048	....	4,863.30
1928..... To Nov. 1	18,614	96,909	......	......	5.21	0.79	....	14,705	2.00	7,352.00
	......	......	1,917	11,752	6.13	....	1.37	2,626	....	1,214.00
								70,301		38,113.05

TABLE 12.—DATA ON DURABILITY TEST SPECIMENS.

Specimen No.	Cement	Real Mix	Slump, inches		Water, gal. per bag	Cement, bags per cu. yd.	Compressive Strength, lb. per sq. in.	
			Design	Actual			7 Days	28 Days
1	Portland	1:6.1	3–4	1	6.67	4.45	1990	3260 3410 2850
2	Portland	1:7.5	3–4	4	7.50	3.65	1110	1750 2680 2030
3	Special	1:7.5	3–4	3½	7.50	3.65	910	1930 2360 1950

Table 13.—Protection with Live Steam, Delray Power House No. 3.

Date of Pour, 1928	Test Cylinder Number	Design Strength, $S=\frac{14000}{8^x}$	Cured with Live Steam under Tarps, hrs.	Air Cured, Protected hrs.	Air Cured, Exposed to Weather, hrs.	Comparative Cylinders Cured in Damp Sand, days	Age When Broken, days	Actual Break, lb. per sq. in.
1-31.........	3825	2500	..	..	...	28	28	4130
1-31.........	3826	2500	..	..	...	28	28	4310
1-31.........	3827	2500	26	18	22	..	2¾	1500
2-2..........	3828	2500	28	24	44	..	4	2070
2-2..........	3829	2500	..	..	...	28	28	4080
2-2..........	3830	2500	..	..	...	28	28	4000
2-6..........	3833	2500	26	17	5	..	2	1010
2-6..........	3834	2500	..	..	...	28	28	4820
2-6..........	3835	2500	..	18	...	1¼	2	800
2-11.........	3840	2500	..	..	...	28	28	3560
2-11.........	3841	2500	..	..	...	28	28	3990
2-11.........	3842	2500	39	..	9	..	2	1570
2-15.........	3843	2900	..	..	...	28	28	4420
2-15.........	3844	2900	..	..	...	28	28	5000
2-15.........	3845	2900	41	..	79	..	5	1430
2-21.........	3850	2500	..	..	...	28	28	4580
2-21.........	3851	2500	..	..	...	28	28	4190
2-21.........	3852	2500	..	..	...	28	28	4600
2-21.........	3853	2500	39	..	13	..	2⅙	1300
2-21.........	3854	2500	36	..	108	..	6	2530
2-27.........	3860	2900	..	..	...	28	28	5030
2-27.........	3861	2900	..	..	...	28	28	4830
2-27.........	3862	2900	24	..	24	..	2	1200
2-27.........	3863	2900	24	..	144	..	7	1690
2-28.........	3864	2500	..	..	...	28	28	3440
2-28.........	3865	2500	..	..	...	28	28	4480
2-28.........	3866	2500	30	..	22	..	2⅙	2040
2-28.........	3867	2500	30	..	162	..	8	2640
2-29.........	3868*	2500	13	..	27	..	1⅔	295
2-29.........	3869†	2500	..	..	...	1	1⅔	717
3-1..........	3870	2500	..	..	...	28	28	4380
3-1..........	3871	2500	..	..	...	28	28	4140
3-1..........	3872	2500	18	..	12	..	1¼	480
3-1..........	3873	2500	18	..	78	..	4	710
3-6..........	3879	2500	12	..	132	..	6	880
3-6..........	3880	2500	12	..	180	..	8	990
3-6..........	3881	2500	..	..	...	28	28	3940
3-8..........	3884	2500	30	..	114	..	6	1690
3-8..........	3885	2500	..	..	...	28	28	3290
3-8..........	3886	2500	30	..	66	..	4	1340
3-8..........	3887	2500	30	..	234	..	11	1190
3-10.........	3889	2500	36	..	60	..	4	1640
3-10.........	3890	2500	..	..	...	28	28	3340
3-10.........	3891	2500	36	..	12	..	2	2120
3-10.........	3892	2500	..	..	...	28	28	4250
3-15.........	3893	2500	16	..	32	..	2	610
3-15.........	3894	2500	..	..	...	28	28	3380
3-15.........	3895	2500	16	..	80	..	4	690
3-15.........	3896	2500	..	..	...	28	28	4140
3-17.........	3897	2500	36	..	12	..	2	2400
3-17.........	3898	2500	..	..	...	28	28	3380
3-17.........	3899	2500	36	..	132	..	7	1900
3-17.........	3900	2500	..	..	...	28	28	3420
3-21.........	3901	2500	12+10	..	74	8	12	2340
3-21.........	3902	2500	..	..	...	28	28	4530
3-21.........	3903	2500	12+10	..	74	10	14	3320
3-21.........	3904	2500	..	..	...	28	28	4130
3-23.........	3907	2500	10	..	38	8	10	2100
3-23.........	3908	2500	..	..	...	28	28	2200
3-23.........	3909	2500	10	14	120	8	14	1850
3-23.........	3910	2500	..	..	...	28	28	3220
3-27.........	3911	2500	..	..	...	28	28	3680
3-27.........	3912	2500	..	..	...	28	28	4220
3-27.........	3913‡	2500	12+60	..	43+29	..	6	3110
3-27.........	3914	2500	12+60	..	43+221	..	14	3540
3-30.........	3915	2500	..	..	...	28	28	4770
3-30.........	3916	2500	60	..	12	..	3	3070
3-30.........	3917	2500	60	..	276	..	14	2780
3-30.........	3918	2500	..	..	...	28	28	4080
10-13........	854	2500	63	..	9	..	3	2230
10-20........	855	2500	30	..	22	..	2⅙	1660
10-20........	857	2500	30	..	114	..	6	2665

* Large stones in cylinders
† In cylinder mould 16 hours.
‡ Outside 43 hours between steaming.

TABLE 14.—COMPARISON OF TESTING MACHINES, DETROIT DISTRICT.

Series	Broken at	Compressive Strength at 7 Days												Average, 6 Cylinders	Difference	
		Cylinder Number	Lb. per sq. in.	Cylinder Number	Lb. per sq. in.	Cylinder Number	Lb. per sq. in.	Cylinder Number	Lb. per sq. in.	Cylinder Number	Lb. per sq. in.	Cylinder Number	Lb. per sq. in.		Lb. per sq. in.	Per Cent
1	A	28 B	*	28 D	*	28 F	442	28 H	584	28 J	495	28 L	778	†575	...	....
1	D.E.Co.	28 A	1083	28 C	1167	28 E	1252	28 G	1240	28 I	1254	28 K	1390	1230	655	57
2	B	14 B	1060	14 D	1120	14 F	1060	14 H	1030	14 J	1000	14 L	1020	1048	...	....
2	D.E.Co.	14 A	906	14 C	977	14 E	1036	14 G	993	14 I	1119	14 K	1006	1006	42	4
3	C	7 C_2	1373	7 C_4	1340	7 C_6	1345	7 C_8	1227	7 C_{10}	1625	7 C_{12}	1301	1368	...	....
3	D.E.Co.	7 C_1	1360	7 C_3	1370	7 C_5	1390	7 C_7	1420	7 C_9	1340	7 C_{11}	1550	1405	37	2½
4	D	7 D_2	855	7 D_4	1208	7 D_6	1125	7 D_8	1035	7 D_{10}	1077	7 D_{12}	1177	1080	...	...
4	D.E.Co.	7 D_1	1360	7 D_3	1270	7 D_5	1250	7 D_7	1330	7 D_9	1300	7 D_{11}	1370	1310	230	17½
5	E	7 A	2070	7 C	1930	7 E	2160	7 G	2170	7 I	2130	7 K	1610	2012	...	....
5	D.E.Co.	7 B	2400	7 D	2090	7 F	2020	7 H	1980	7 J	2340	7 L	2210	2170	158	7½

* No load.
† Four cylinders.

The Concrete Lining of Detroit Water Tunnels of Detroit, Michigan

By L. G. Lenhardt*

This paper is to present the design and construction of the concrete lining of the land tunnels now under construction for the Department of Water Supply, of the City of Detroit. These tunnels when completed will be about 11 miles in length, cost about $5,700,000, and involve the use of about 200,000 yds. of concrete. The purpose of these tunnels is to convey raw water from the shore shaft to water plants some distance removed. The entire water project when completed will cost about $30,000,000 exclusive of the distribution system and will provide a system independent of the present plant and of about 50 per cent greater capacity.

Upon first considering the problem of lining the tunnels it became apparent that an ideal lining must be durable, of great strength and of maximum smoothness.

Durability or permanence was of prime importance as the failure of any small section of tunnel would place a large part of the city without water until the failure could be repaired. The scarcity of available routes weighed against duplicate lines and the depths involved (70 to 110 ft. through the heart of the city) precluded speedy repair.

Strength of lining had to be considered not only from the standpoint of the design loads but also from the standpoint of unusual or unexpected loads. Soil conditions at the depths involved were uncertain at best as no tunnels had been built as deep locally. Unbalanced and unusual loading, which can not well be designed for, also arise frequently in underground construction. Further, the lining must be strong enough to take grouting pressures which may be as high as 100 lb. per sq. in. over localized areas.

Smoothness of lining was highly important because of the limited head available and the fact that a foot of head was evaluated at $2500 per year. On this basis the head loss between ordinary construction and high quality work could be capitalized at $270,000.

A careful study of the conditions to be encountered lead to the assumption that the local blue clays through which the tunnels would be driven had angles of repose at the depths involved of not in excess of 6 deg. It was also believed that the elastic ring method would give the most accurate determination of the lining stresses. As grouting

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pressures and unbalanced loadings might cause heavy stresses in almost any part of the section and as the head available must be conserved, the circular section was chosen as being the best adapted both structurally and hydraulically.

It became evident that there was no satisfactory information about the strength in comparable arches of the various materials of which a tunnel lining might be constructed. At that time, early in 1925, there was but meager information about the concrete aggregates available locally. Cements were being bought to a large extent on what might be termed prejudice. Little use had been made of hard burned brick and while both concrete and common brick had been extensively used in the sewer tunnels it was difficult to make comparisons. In view of these considerations it was decided the only satisfactory way these

FIG. 1. HALF-SIZE TUNNEL SECTIONS IN POSITION FOR LOAD TESTS.

questions could be settled was to make a thorough investigation and build a series of arches of the same size and shape of different materials and test in various ways.

Sections 6 ft. in diameter, or one-half that of the 12-ft. tunnels were adopted as standard for the test. These were built 4 ft. in length and with 8½-in. wall thickness (equivalent to two rings of brick). Sections were constructed of local common brick, of local concrete aggregate with different mixes, and of 2½- and 3-in. vitrified shale brick. Altogether 14 such sections were built. The general layout is shown in Fig. 1.

Loads were applied over the top quarter of the arches by means of a sand box and pigs of lead. This method of loading is similar to that adopted by the A.S.T.M. for testing sewer pipe up to 42 in. in diameter. All sections were built in place by carefully shaping out a trench to the springing line. Where support over the bottom quarter only was desired, the earth was dug away to the bottom quarter after

the sections had set. Sections were thus tested with loading over the top and bottom quarter, and also loading over the top quarter and embedment to the springing line.

Wherever possible, Ames gages reading to one 1000th part of an inch, were used for measuring both horizontal and vertical deflections. Settlement of the arch as a whole was checked by means of a piano wire stretched through the arch about 18 in. from the invert. The loadings at which cracking and failure took place were noted, as well

FIG. 2. METHOD OF LOADING TUNNEL TEST SECTION.

as the general behavior of the arch during the test. After a test had been completed the arch was carefully examined. The method of loading and the use of the Ames gages are shown in Figs. 2 and 3.

A typical load-deflection curve is shown in Fig. 4 which is for a concrete arch supported on the bottom quarter and loaded over the upper quarter. The concrete in this arch showed a compressive strength of 3000 lb. at 28 days. On Fig. 5 is shown the deflection curve for a concrete arch reinforced only over the upper half and with loading over the upper quarter and cradled in clay to the springing line. The con-

crete showed a compressive strength of 2350 lb. at 28 days as a higher water-cement ratio was used in order to work the concrete around the steel. A study of the loading on this arch shows that it should have failed at the invert considerably before final load was reached if the customary assumption of uniform loading over the bottom half held. No evidence of failure was observed so it must be assumed this assumption is conservative.

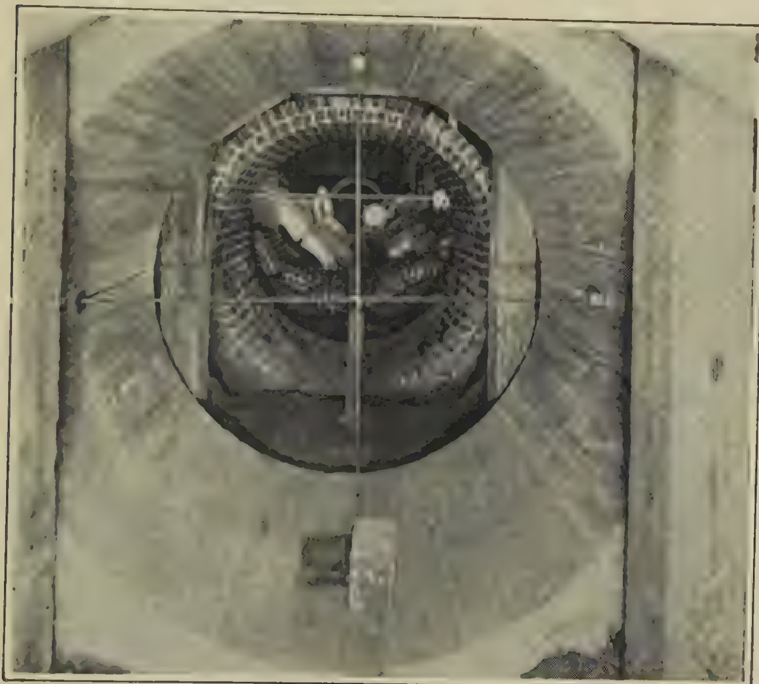

FIG 3 ARRANGEMENT OF AMES GAGES FOR MEASURING HORIZONTAL AND VERTICAL DEFLECTIONS.

All materials used were subjected to standard A.S.T.M. tests at the city laboratory. Samples of the concrete entering into the work were taken both in the form of cylinders and 8 x 8 x 32-in piers. Similar piers were also made of the brick and mortar used in the arches.

Samples were secured of most of the commercial aggregates and cements in the local market and a large number of tests made using various mixes, cements and water-cement ratios. Particular emphasis was paid to securing workable mixes. It was realized that tunnel conditions require an unusually workable mix as the invert concrete must flow into place without an excessive amount of water while at the crown

where spading and ramming are difficult, a comparatively dry concrete must be used.

It was found that among the concrete arches those in which an oversanded mix was used showed a much smoother surface than where a normal amount of sand was used; further that the concrete was of

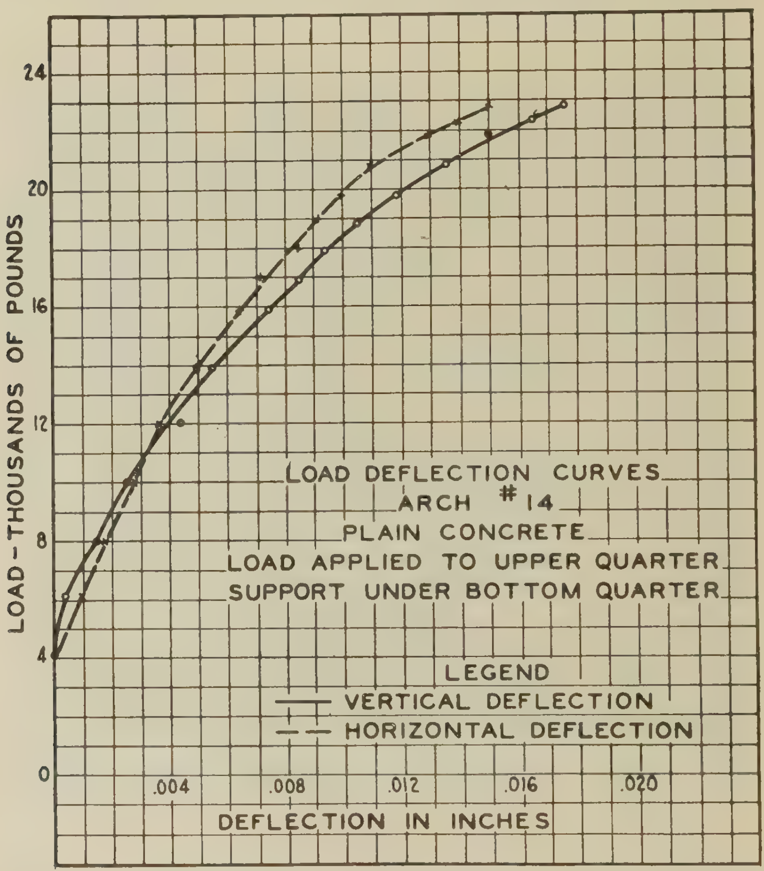

FIG. 4. TYPICAL LOAD-DEFLECTION CURVE.

more uniform quality as determined by compressive tests of cylinders. An independent series of tests showed that by keeping a constant water-cement ratio somewhat greater slumps and strengths could be expected by oversanding the mix. These greater strengths were accompanied by more uniform results. In general it was found that for Detroit aggregates a mixture of two parts of sand to three parts of coarse aggregates

gave the best results. A study of costs revealed that there was a surprisingly small difference in costs between the richer, oversanded mixes and the so-called standard 1:2:4 mixes, a 1:2:3 mix being found to cost but 12 c. per cu. yd. more than a 1:2:4 mix. This small difference is due to local conditions, cement and sand being low in cost compared to coarse aggregate.

The importance of water control was realized early in making the tests. Previous to this time, 1925, no important work had been attempted locally using complete water control and it was hoped that sufficiently accurate results could be obtained by the slump control. However, the enormous strength variations appearing in quite comparable mixes

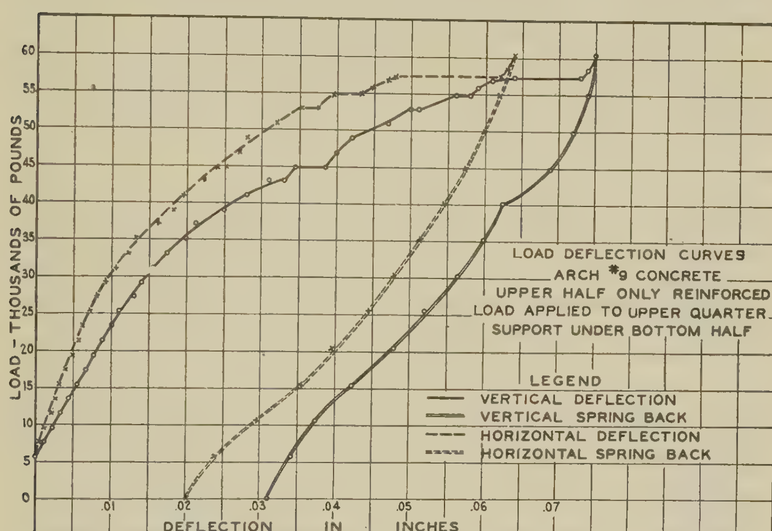

FIG. 5. LOAD-DEFLECTION CURVE FOR A SPECIAL TEST TUNNEL SECTION CRADLED IN CLAY TO ITS SPRING LINE.

of the same slump speedily led to an appreciation of water control and the necessity for its use on construction.

During the tests cement was usually used as soon as received from the warehouse. However, some cement was stored from two to five months in the cement storage building of the department. This building was of brick construction with the floor elevated 3 ft. above grade. Cement was placed on planks laid on the concrete floor. Storage conditions were better than anything that could be found on a construction job, yet marked reductions in strength were found. For 5 months' storage about 30 per cent reduction in strength was obtained and even at 2 months reductions were marked. This pointed to the necessity for limiting storage of cement on the job.

A study of the various cements used showed considerable difference in strength between different brands, one brand in particular at that time showing to marked disadvantage. It was found that this brand barely passed specification requirements for tensile strength and further showed efflorescence on setting. This latter quality had caused it to have a rather limited use generally for exterior walls. Efflorescence would increase the coefficient of roughness in the tunnels, and was therefore very undesirable. As a result it was decided that the tunnel specifications should state that no cements showing an efflorescence upon setting could be used on the work. It was further decided to raise the tensile strength requirements 25 lbs. higher than the then prevailing A.S.T.M. Standard in order to get the higher grade cements.

By the time the tests were finished the Concrete Committee of the Detroit Engineering Society had issued some recommended specifications for concrete aggregate. These specifications were a long step ahead of the customary specifications previously in vogue. It was decided to use these specifications in so far as practicable in compiling specifications for the tunnel work.

A study of the tests and other data that had been collected resulted in a recommendation that vitrified brick and concrete lining be placed in the specifications on a parity and that whichever material proved to be the lowest in competitive bidding should be used in the tunnels. The safe working strengths recommended were 667 lb. per sq. in. for both vitrified brick and concrete masonry. On the basis of these strengths, wall thicknesses of 21 in. were recommended for the most of the 12-ft. tunnels and 25 in. for the 14-ft. tunnels. It was decided that it would be better to use an unreinforced section in so far as possible using reinforcement only in extreme conditions because of the many difficulties occasioned by its use in tunnel work.

With the foregoing as a background the specifications for the concrete were written, the essential points of which are as follows:

Cement:

On the first specifications, the C 9 – 26 T, A.S.T.M. specifications were specified. These tentative specifications had come out in the meantime and were found to agree with the conclusions previously arrived at. The conclusions for efflorescence as previously stated were included and a maximum of 20 days storage was specified.

Fine aggregate:

Fine aggregate was defined as that passing a No. 4 sieve and retained on a No. 100 mesh sieve. An average fineness modulus of 3.00 was specified with an upper limit of 3.25 and a lower limit of 2.75. The grading was required to be uniform from fine to coarse.

Coarse aggregate:

Coarse aggregate was defined as that passing a 2-in. square opening and retained on a No. 4 sieve. The average fineness modulus specified

was 7.25 with an upper limit of 7.60 and a lower limit of 6.90. A uniform grading from the upper to the lower limit was further specified.

Proportion:

The mix was specified as 1:2:3 with 6½ gal. as the maximum amount of water per sack of cement, inclusive of contained moisture in the aggregates. The proportions of aggregate stated were nominal, the exact division between fine and coarse being such as to produce a concrete of the desired workability using the least amount of mixing water. The sand was required to be measured in an inundated or submerged condition and the excess water required measured in a batch tank. Coarse aggregate was required to be measured in either volume or weighing hoppers.

Mixing:

The time of mixing was specified as 90 sec. after all materials were in the mixer during which time the peripheral speed of the mixer should be about 200 ft. per minute.

Forms and surface finish:

Forms were required to be made of metal and their design to meet the engineer's approval. Rigid requirements as to surface finish were adopted, notabie among which was requiring the use of rubber-faced hammers for vibrating the forms. As hand-handled forms would undoubtedly be used, rigid requirements as to removal of fins between plates were set up.

Grouting:

All voids and excess excavation outside of the neat lines of the tunnel line were required to be filled with grout through grout pipes provided for the purpose.

Bids for the first two sections of the work were received in October, 1926, and contracts for the remainder of the work were let at intervals until October, 1928, when the last piece of tunnel was placed under contract. All told there will be seven sections of tunnel, the smallest contract amounting to close to $600,000 and the largest contract close to $1,400,000. Three different contractors, all local, have secured all the contracts although 29 contractors of whom 16 were from outside of the city have submitted bids. To date, November, 1928, 38,276 ft. of 12-ft. tunnel and 4681 ft. of 14-ft. tunnel involving the placing of 138,312 yd. of concrete have been completed. The price of the concrete (all specifications identical) has steadily decreased as shown in the following tabulation:

SECTION	1	2	3	4	5	6
Bid Price	23.30	21.70	16.00	15.85	14.25	14.00

It is probably safe to assert that although the decreasing price is partly a reflection of the increased competition for the work, that the

confidence the contractors now have in securing satisfactory concrete under the specifications has a great deal to do with the descending scale of prices.

Before any contracts were let, however, the importance of good inspection in securing the results contemplated was realized. Nation-wide competitive examinations were held for the positions of field engineer and inspector. In the examination concrete control was stressed quite heavily. The nucleus of the field organization was assembled considerably before construction work was started and given a thorough drill in the specifications and what were regarded as fundamentals. A concrete laboratory was established and field engineers and inspectors required to attend a short course in the concrete laboratory until it was felt they were thoroughly versed in the rudiments of concrete control. This course consisted in making screen analyses of various aggregates, slump tests, the making of concrete cylinders with various water-cement ratios and in training in all concrete control work that might be needed in the field.

It was the feeling in the department that cement inspection could probably best be handled by a private testing laboratory for four reasons. First: the most important reason was that a private testing laboratory would represent an absolutely impartial body whose results would not be open to question in case trouble should arise. Second: not only was the department ill equipped to handle cement testing but men were lacking who had the necessary training to satisfactorily make cement tests. Third: after some study it was found that it would cost less to have the cement testing done by a private laboratory than to do the work in the department. Fourth: such testing as might be done in the department would supplement and further check the results from the private laboratory and thus lead to absolute confidence in the cement.

Cement tests have been made at the testing laboratory throughout the work. As all cement was secured from one company a sealed bin was set aside and none but cement from this bin went into the work. All tests were made of bin samples secured according to the standard A.S.T.M. methods. To date none of the cement has failed to pass specifications and in fact the briquette tests have consistently showed well over the minimum A.S.T.M. requirements. Supplementary tests by the department laboratory have checked the results of the testing laboratory on the briquette tests although but comparatively few of these checks have been made. The chief work of the department laboratory has been to make concrete cylinders from time to time with the aggregates and water-cement ratios used on the work. The concrete has consistently showed strengths of 3200 lb. at 28 days with 6½ gal. of water and of the mix and materials used in the tunnels. This is considerably above what had been anticipated from the Abrams curve as the 6½ gal. water ratio on the $\frac{14{,}000}{7^x}$ curve gives about 2650 lb. per square inch.

From the very beginning of the work, inspectors have been placed at the pits whose sole duty it has been to see that none but inspected aggregates are shipped for use on the work. After considerable experimenting with samples it has been found that the most uniform and consistent results were secured by taking samples from the loading belt rather than car samples. Screen analyses are made at the pits and after the car is determined as satisfactory a tag is tacked on the car. The type of tag is shown on Fig. 6. To date 118,000 yds. of gravel and 84,300 yds. of sand have been accepted as satisfactory. This represents 3470 cars of gravel and 2218 cars of sand. The screening curve for all aggregate shipped to date is given in Fig. 7, which shows the uniformity with which the average of the specifications has been approached.

DIVISION OF ENGINEERING

BOARD OF WATER COMMISSIONERS
OF THE
CITY OF DETROIT

BUREAU OF LAND TUNNELS

INSPECTED

MATERIAL________________ CAR NO.________________

PRODUCER________________________________

INSPECTION REPORT NO.______ SENT TO________________ ON________

DATE SHIPPED________________ SIGNED BY________________

NOTE: THIS CARD IS TO BE REMOVED AND RETAINED BY THE ENGINEER'S REPRESENTATIVE

CONTRACT________________ DATE RECEIVED________________

FIG. 6. INSPECTOR'S CARD TO BE TACKED TO APPROVED CARS OF AGGREGATES.

Upon receipt of the cars in the material yards the tags are removed by the yard inspector who sees that none but inspected cars are loaded onto the storage piles set aside for the purpose. The yard inspector further sees that proper methods of unloading are used. Where clam shells are used to unload the cars it has been found that dumping while the bucket is swinging gives the least segregation. As this is also the fastest method of unloading little difficulty has been encountered in securing the co-operation in the supply yards. The yard inspector further sees that none but inspected material is loaded out to the work and he further supervises the loading of the trucks. A special effort is made at the yards to secure a uniform cross section from the storage piles.

On receipt of the aggregate on the work a further effort is made in securing uniformity, as all jobs are equipped with overhead bins of from 200 to 250 tons capacity, segregation can easily occur. There have been two methods of loading the bins: one by clamshells placing

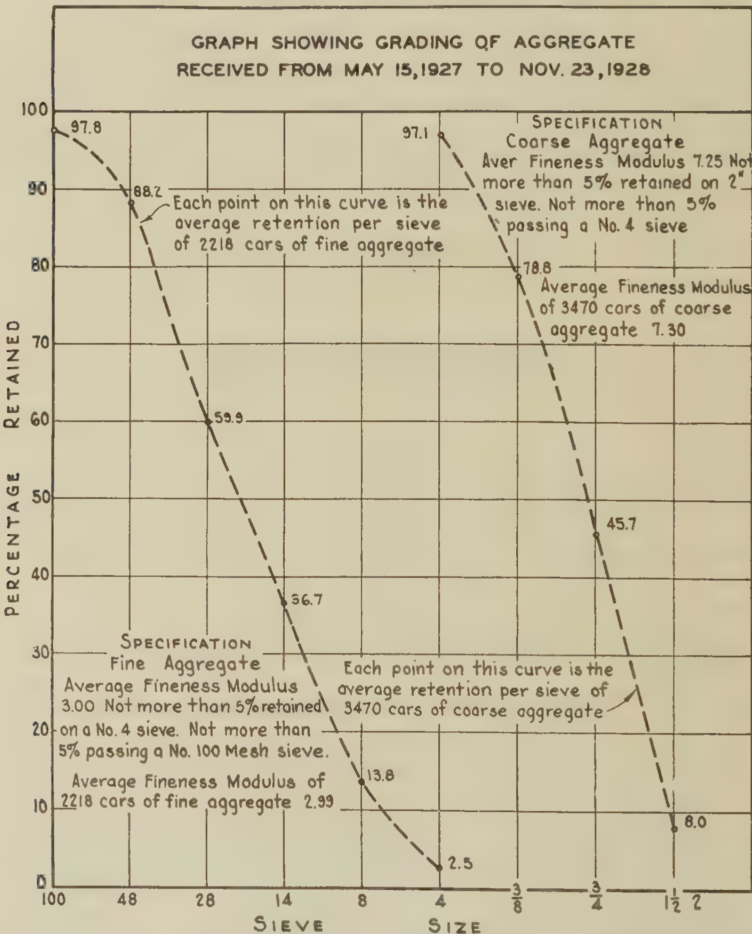

FIG. 7. SCREENING CURVE FOR AGGREGATES.
Note close agreement with specification.

the dumped material in the bins and the other by trucks dumping into hoppers with the aggregate being hoisted to the bins by bucket conveyors. With clamshell bin loading it has been quite simple to secure a uniform aggregate in the bins as the clamshell operators have been

instructed to load alternately from the outside and middle of the dumped material. With the bucket conveyors uniformity has not been so easy to obtain. It has been found that the best results can be secured by moving the chute on top of the bins repeatedly during the loading operations and having a laborer, who is stationed on top of the bins during the operations, level up the aggregate. This has secured fair results although

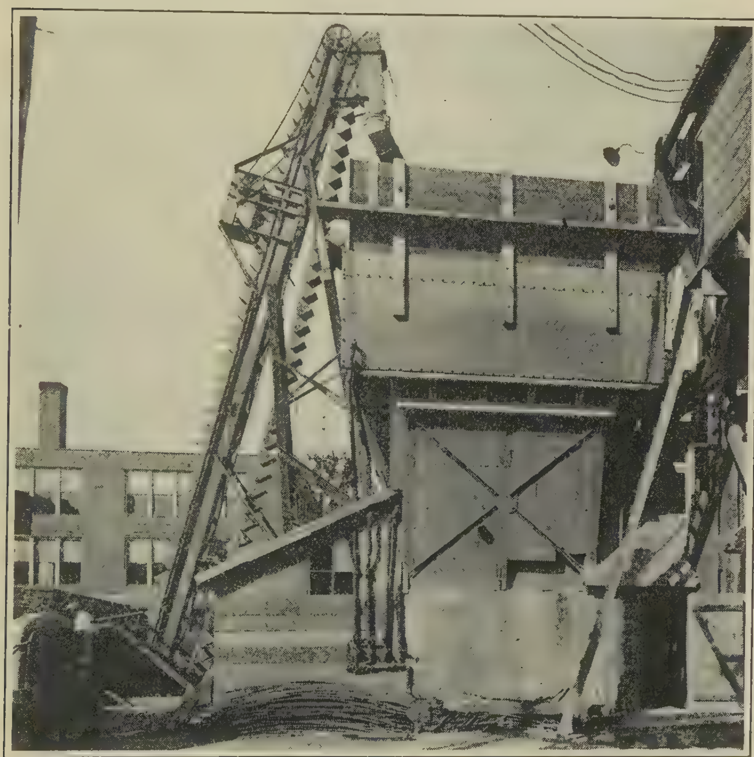

FIG. 8. CHARGING AGGREGATE BINS WITH BUCKET ELEVATOR.

they have not been as uniform as when the clamshell method of loading is used. Prior to hand leveling in the bin when loading it was repeatedly found that with average specification material with a fineness modulus of 7.25 that the coarse aggregate coming out of the bins first had a fineness modulus of about 6.90, the lower limit of the specifications, and the aggregate last out of the bins a fineness modulus of 7.60, the upper limit of the specifications. This was undesirable not only from the stand-

point of the lack of uniformity but from the fact that this meant that the finer material went into the construction of the invert and the coarser material was used in the crown, whereas the best results were obtained with the conditions reversed. However, by paying close attention to bin loading it has been found possible to cut down variations in fineness modulus of the coarse aggregate to not to exceed 0.30.

One field engineer has been placed in charge of each section of the work, depending somewhat on the job and there have been assigned to

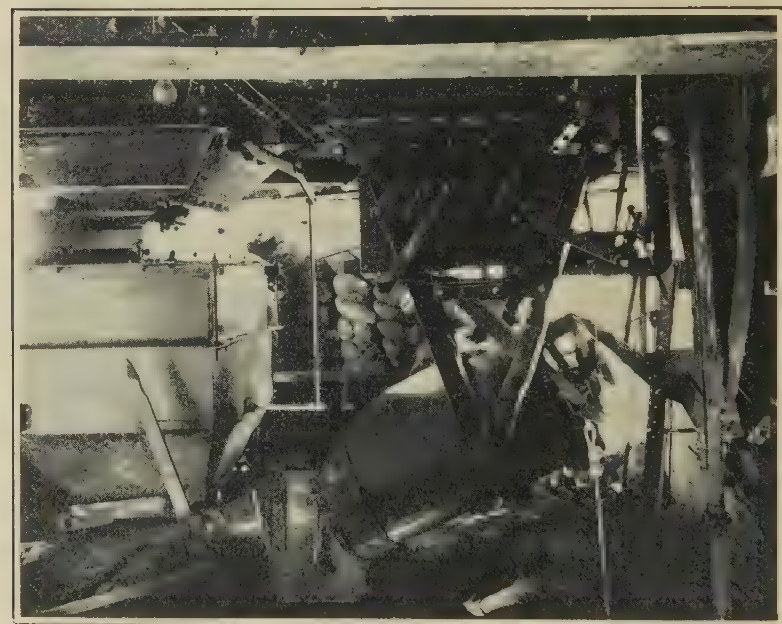

FIG. 9. ARRANGEMENT OF INUNDATOR, WATER MEASURING TANK AND AGGREGATE HOPPER.

him, usually, five inspectors, of whom one is placed on excavation, one on office and record work and three on concrete control. One of the concrete inspectors is assigned to each heading and one to the mixer. The mixer inspector checks the inundator, batcher and water settings, the time of mix and the cement. The heading inspectors see that the concrete is properly deposited in the forms and well spaded. The heading inspectors also make slump tests and cylinders from each day's work. At least one cylinder a day is made from the invert, springing line and crown concrete. These cylinders are let stand in the tunnel until the

next day when they are removed from the molds and placed in the sand curing box which is also kept in the tunnel. Cylinders are not removed from the damp sand until about 24 hr. before being tested. In this

FIG. 10. VIBRATING THE TUNNEL FORMS WITH A FOUNDRY SAND-RAMMER EQUIPPED WITH A HEAVY RUBBER TIP.

manner a complete record of 28-day strengths for each day's operation is kept.

There has been a great similarity in the contractor's plants used. In every case a mixer of approximately ¾ yd. capacity was placed in a

pit with the top of the mixer below ground level. Over the mixer were the inundator, the water measuring tank and the coarse aggregate hopper as shown in Fig. 9. The aggregate is placed in bins of from 200 to 250 tons capacity feeding directly into the hoppers by gravity. All mixers are equipped with timers which ring a bell when the 90-sec. mix has been secured.

The concrete is discharged from the mixer into a baffled chute which extends to the foot of the shaft usually about 6 ft. above the rails. The concrete is thus dropped directly into the cars, each of which is of sufficient capacity to take the entire mixer charge. These cars are all of the side dump type with a V-shaped body. The concrete is transported to the heading in train loads of from 3 to 8 cars, depending on the distance of haul, whether compressed air is being used necessitating going through the locks, the grade of the track and the like. Concrete below the springing line is placed by the cars being pulled open up an elevated platform at the springing line and dumped on large pans which slope toward the forms. It can be seen that the invert concrete must readily flow into place with such assistance as may be secured by vibrating the forms. Above the springing line the concrete is hand placed, the cars being dumped on the platform and the concrete being shoveled in. As each successive row of plates is filled an additional row is fastened in place and concrete shoveled in. This continues until the top portion of the arch is reached when concrete must be shoveled in from the end, working back from the old concrete toward the far end of the heading.

As soon as concrete shows in the forms near the springing line hand spading is resorted to as well as vibrating with the rubber-faced air hammers. It is very difficult to use the vibrators at the crown as the crown plate of the forms is not pinned in, but is held by wing nuts. Too much vibrating loosens the plates and hence ramming from the end forms is the chief reliance in securing dense crown concrete. The vibrators referred to consist of the conventional sand-rammer as used in foundry practice. This was adopted after some experimenting with various types of air tools. A heavy rubber tip is placed on the striking end of the hammer. The vibrator and its use is shown in Fig. 10.

All forms used were manufactured by the Blaw-Knox Company and consist of 8-in. ship channels rolled to the tunnel circumference over which plates are pinned. The channels are in four pieces and need pinning together to form the ring. Plates are 36 in. long and are fastened to the ribs by pins.

While an unusually tight fit for hand-handled forms was obtained, nevertheless some fins and ridges were left at the joints between the plates, particularly at the vertical joints. Longitudinal joints were very smooth for the most part and gave but little trouble. The forms on the whole were very satisfactory considering the various conditions of usage. Unusually heavy ground pressures occasionally would distort the plates making repairs necessary but under normal conditions the plates stood up well. Careless handling was probably responsible for

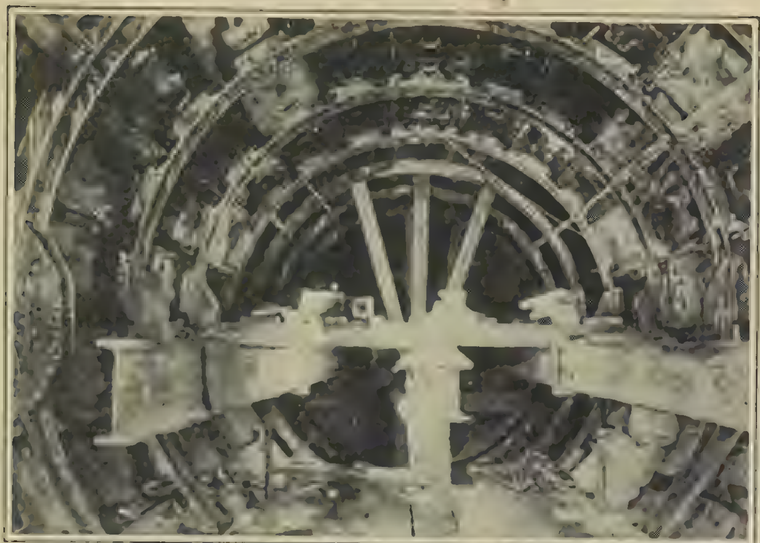

FIG. 11. ERECTING FORMS AT TUNNEL HEADING.

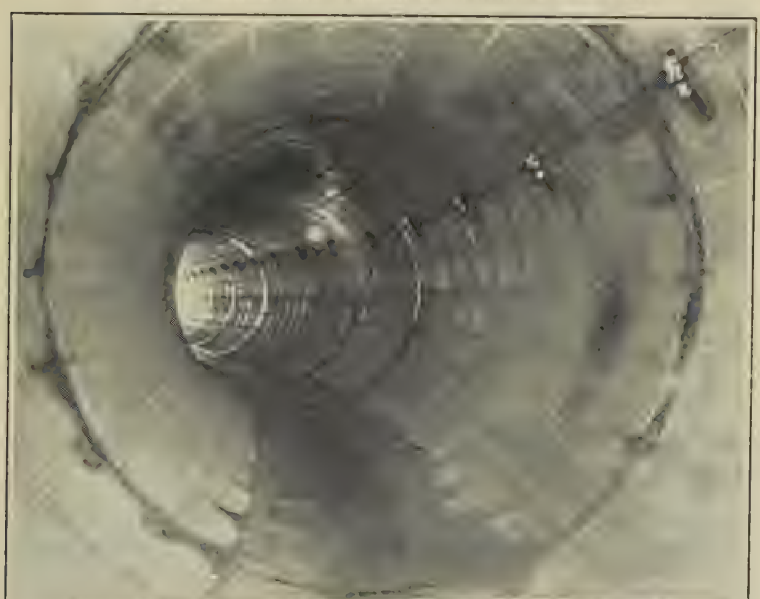

FIG. 12. A COMPLETED CURVE IN THE WATER TUNNEL.
Straight forms were used for all curves and the consequent projecting lugs of concrete, the thickness of a form plate, were removed with bushing hammers.

more trouble with the forms than any other cause. In stripping it was customary for the men to throw the plates down and they very frequently landed on the corners, bending them. This meant that the heading inspector had to check the form setup very carefully to determine which plates should be culled. The combination of the severe handling and heavy loads on the forms above the springing line was evidenced to some

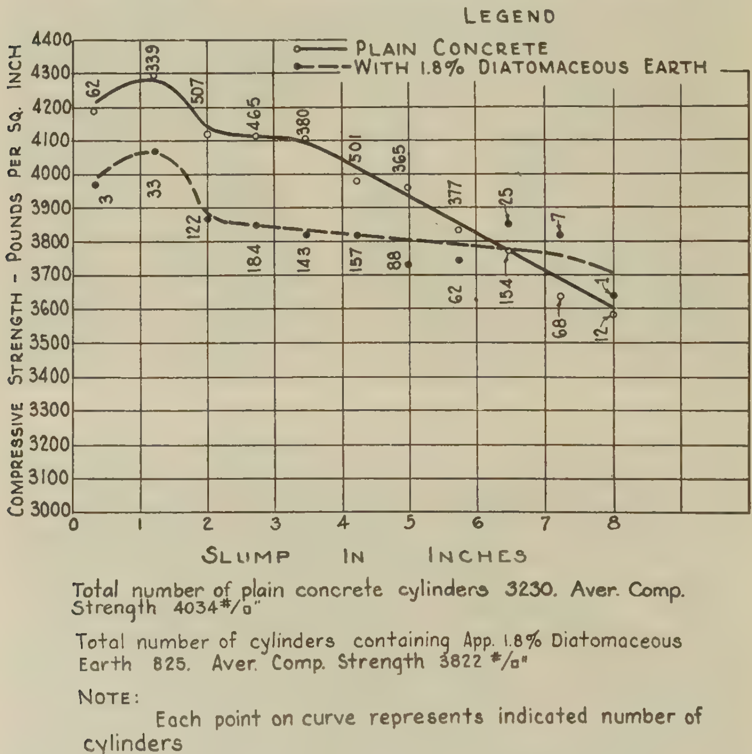

FIG. 13. RELATION BETWEEN COMPRESSIVE STRENGTH AND SLUMP OF A 1:2:3 MIX.
Strength at 28 days. All data from daily field tests.

degree in the surface finish of the tunnel, the finish of the surface below the springing line being on the whole better than that above.

All curves in the tunnel were built with the standard forms. It was determined that with a radius of 180 ft. that the plates on the outside of the curve would have sufficient bearing on the ribs to stay securely in place. This left a projecting lug of concrete of the thickness of the form plate which had to be dressed down when the forms were stripped, but the finishers did this so skilfully that very smooth curves were obtained.

As the work developed it was found that the combination that gave the greatest strength and smoothest finish at the invert was concrete with a slump between 5 and 6 in., at springing line 4 in. and at the crown 2 in. Due to the uniformity of the aggregate the heading inspectors were enabled to gauge the water content with a considerable degree of accuracy by means of the slump tests and could keep the mixer inspector informed as to the desired water content by means of the telephones which were placed in each heading. The 5- to 6-in. slump corresponds roughly to a 6–6½ gals. of water per sack of cement and the 2-in. slump to about 5–5½ gals. of water per sack of cement. While a concrete with a greater slump would have been desirable in the crown, nevertheless the exigencies of hand placement demanded a fairly dry concrete. The result has been that part of the inspectors' duties has been to see that sufficient water is being used in the crown concrete as the tendency of the contractors' men has always been to dry up the mix beyond the limit with which good placement could be secured. The curve (Fig. 13) was determined by the daily field tests on the tunnel work.

The average strength of all the concrete placed in the tunnel work to date is 4014 lb. per sq. in. at 28 days, the test cylinders being made from concrete taken from the forms. The strengths on the various sections are as follows:

Section 1	4020
Section 2	3910
Section 3	4180
Section 4	3820
Section 6	3956

No concrete has been placed as yet on Section 5.

A test was made on Section 2 to determine the relation of the concrete in the forms to the concrete at the shaft with the following 28-day strengths being reported.

At mixer	3531 lb. per sq. in.
At bottom of chute	3593 lb. per sq. in.
From forms	3872 lb. per sq. in.

A five-day run on Section 4 to determine the effect of haul on the concrete is of interest. The haul in this case was about 4000 ft. and all cars had to be passed through the air locks making a considerable delay The concrete at the bottom of the chute showed 3344 lb. per sq. in. at 28 days and the concrete from the forms showed 3350 lb. per sq. in. at 28 days. It is quite evident from the above figures that there was little segregation occurring from the time the concrete left the mixer until final deposition in the forms.

On Fig. 13 is shown a separate curve of relation of strengths to slumps on concrete with an admixture of diatomaceous earth. As the work had progressed so that the headings were some distance removed

from the shaft on Sections 1 and 2 the contractor who had both of these sections felt that the concrete was not dumping as freely from the cars as might be desired. He requested permission to use a diatomaceous earth admixture in order to get a free dumping concrete. As some tests had been made previously in the laboratory which indicated that this admixture had at least no deleterious effects, permission was granted

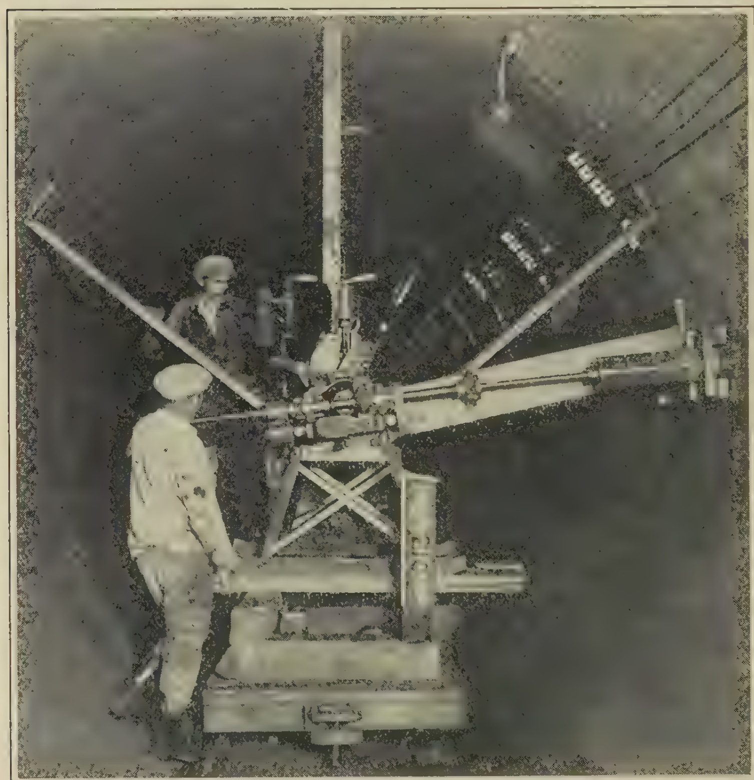

FIG. 14. DIAMOND DRILL SETUP FOR TAKING A SPRINGLINE CORE.

him to use not over 2 per cent of the admixture by weight of the cement used at no extra cost to the department. The contractor used this admixture throughout the balance of these two sections in the proportion of 1.8 per cent by weight and felt that it gave him worth-while results. The lower strengths reported for the concrete with admixture are probably a reflection of the inspecting forces' attitude to try to keep a uniform slump. As the admixture absorbs water quite readily, it is quite probable

that more water was used than with the plain concrete and hence the slightly lower strengths. The flatness of the slump-strength curve is especially noteworthy and probably indicates the absorptive power of the admixture. A comparison of 402 cylinders made of the concrete on Section 2 prior to the use of the admixture and the same number of cylinders following its use show strengths of 3924 lb. per sq. in. at 28 days as compared with 3902 lb. per sq. in.

Early in the work an attempt was made to get a comparison of 7- and 28-day cylinders with a view to determining whether 7-day tests would give a reliable indication of the character of the concrete. Earlier tests were advisable if possible as the tunnel might have progressed 600 ft. before strength tests became available if 28-day tests were used. Comparisons made between 239 cylinders each of 7-day and 28-day concrete showed the following:

Average strength at 28 days........... 4055 lb. per sq. in.
Average strength at 7 days............ 2786 lb. per sq. in.

Following the form of Slater's formula the following formula was arrived at:

$$S_{28} = 24\sqrt{S_7} + S_7$$

However, a comparison of the identical concretes at 7 and 28 days revealed wide discrepancies, comparable concretes having the highest 28-day strengths showing the lowest 7-day strengths and with almost the opposite results. Accordingly the 7-day cylinders were not used for control except in special cases.

While it was felt that the curing conditions in the tunnel were favorable owing to the contact of the outside of the lining with moist clay and the moist air and condensation on the inside, nevertheless there seemed to be a question as to whether damp sand curing properly represented the concrete in the tunnel. Accordingly a series of tests were made with cylinders of tunnel concrete cured under the following conditions: (*a*) on the tunnel invert under the working floor where there was always considerable dampness; (*b*) in damp sand; (*c*) exposed to the weather on the surface; (*d*) in water; and (*e*) with the cylinders half buried on their sides in clay, the other half exposed to the air. This last named condition was believed to simulate tunnel conditions. The cylinders were let stand in the moulds for 24 hr. and cured as above outlined for 27 days and then tested. The compressive strengths obtained were as follows:

(*a*) 4281 lb. per sq. in.
(*b*) 4340 lb. per sq. in.
(*c*) 4193 lb. per sq. in.
(*d*) 4389 lb. per sq. in.
(*e*) 4720 lb. per sq. in.

The result for series (e), those cylinders exposed to the weather, are open to suspicion as some sand and dirt was found around the cylinders. However, series (b) and (d) agreed very closely as might be expected. As series (c) showed the greatest strengths there seemed to be no question but what sand-cured cylinders would not exaggerate the strength of the concrete in place.

FIG. 15. CORES TAKEN FROM CROWN OF TUNNEL.
Some honeycombing is visible. Note grout. See Note under Fig. 16.

The joints between adjacent days' work were shown in the plans as of the tapered keyway type, the depth and width of the keyway being each one-third the thickness of the tunnel wall. A 1-in. taper on each side of the keyway proved satisfactory. As butt joints are usually used on tunnel work it was hoped the keyed joint would eliminate the usual joint troubles of cracking and leakage. Before the fresh concrete was

placed the surface of the old concrete was brushed with a wire brush, wet down thoroughly, and neat cement paint brushed on. Little or no trouble was experienced with the joints opening while near the shafts. As the headings became farther advanced some joints began to open, usually not exceeding $\frac{1}{16}$ in. at the crown and running out to a hair crack at the springing line. It was seldom any evidence of cracking was found in the invert. The cracking usually became more noticeable during warm weather. Few cracks were found closer than 200 ft. to the heading.

From the circumstances it appeared that the primary cause of the cracking was the too rapid contraction of the upper portion of the arch due to more rapid evaporation being experienced there than from the lower half. Accordingly the surface of the concrete was thoroughly wet down twice a day for a distance of at least 400 ft. back from the forms. This eliminated a large portion of the cracking but some crown cracking still persisted and such cracks as did occur were now about $\frac{1}{8}$ in. in width or twice the previous size. Hooked steel dowels of $\frac{3}{4}$ in. round stock placed 3 ft. on centers were then used in the crown and in conjunction with the wetting practically eliminated joint openings although hair cracking still occurred to some extent.

Where cracking or leakage occurred at the joints pressure grouting was always resorted to. At times grouting from one hole would stop leakage at a joint but in some cases as many as 30 holes were drilled at a joint. The joints in which a large number of holes were drilled did not necessarily take more grout than where a smaller number of holes were drilled but the grout seemed to be better distributed and hence stopped leakage. Where the joints had opened to any considerable extent they were opened sufficiently farther to allow lead wool to be thoroughly caulked into the joints and cement mortar was then rubbed onto the surface.

It was found that the cement mortar used in smoothing up joints bonded exceedingly well to the tunnel lining. The procedure in placing this mortar was to first wet down the surface and wait until the free water disappeared and then rub on the mortar. When the mortar had set sufficiently cement dust was rubbed over the surface until smooth. In several places it became necessary to make cuts through such mortar and in no place was the mortar broken from the concrete. Several attempts were made to break this mortar from the concrete by hammering but the bond was such that the mortar could be loosened only by removing a portion of the concrete with it.

When the first strength tests on the 28-day cylinders began coming in and unusually high strengths for this city were shown (averaging 4000 lb. per sq. in.) there was some doubt expressed as to whether the cylinder concrete truly represented the concrete in place. The difference in curing conditions, the difficulty of securing dense concrete with the slumps that were used, the fact that the cylinders were rammed considerably more than job concrete would be and the possibility of inspectors, unconsciously perhaps, making the cylinders out of the best concrete in

the forms all were mentioned as tending to favor the cylinder concrete as against the concrete in place. There was further a desire in the department to ascertain various other facts in connection with the work such as tightness of joints, the effect of the grout and the closeness with which

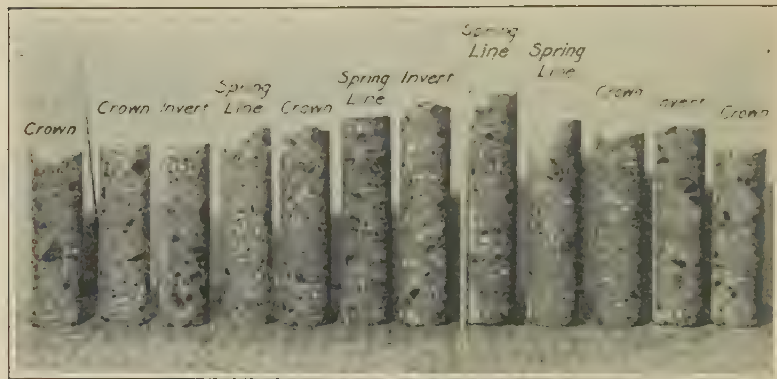

FIG. 16. DIAMOND DRILL CORES TAKEN FROM COMPLETED TUNNEL LINING. Note better appearance of crown cores than in Fig. 15 on account of increasing slump to about 2 inches.

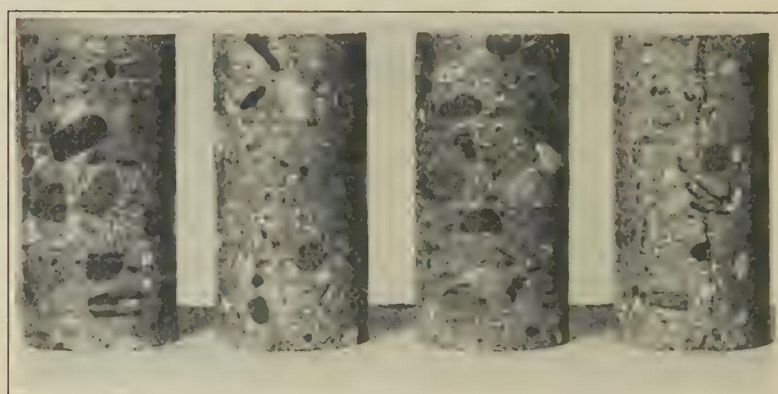

FIG. 17. CORES SAWED INTO 12-IN. TEST LENGTHS.

the inspectors' reports on thickness agreed with the actual thickness in the lining. After considerable investigation it was finally decided to drill a series of 6-in. cores in the tunnel lining by the diamond drill method. As there was no precedent by which to measure diamond loss it was found difficult to induce drillers to submit proposals. However, a drilling company was finally obtained who agreed to take out the cores on a

daily rental basis for the machine, drill runner and diamond setter, and they were further to bear all diamond loss except where steel was encountered.

The core drilling had not progressed very far before it was evident that the crown concrete placed with slumps of 1½-in. and less was not satisfactory. While there was no evidence of honeycombing at the surface, nevertheless some honeycombing would be found in the interior of the concrete as shown in Fig. 15. Instructions were immediately issued on the work to use as much water in the crown concrete as practicable, which meant slumps of about 2 in. This improved the quality of the crown concrete considerably as was evidenced by the later cores shown in Fig. 16, but there was still evidence that the crown concrete was not as dense as that placed with the higher slumps. The concrete from the springing line and invert quite consistently showed weights of 155 lb. per cu. ft. whereas the crown concrete, even where apparently sound, ran from 3 to 5 lb. per cu. ft. under this. The cores were sawed into 12-in. lengths for test purposes as shown in Fig. 17. Comparisons of the strengths between the invert, springing line and crown cores also revealed that the wetter concrete gave somewhat higher strengths in place as is shown by the following:

Average Strengths of Concrete Cores at All Ages

Invert	5040 lb. per sq. in.
Springing Line	4980 lb. per sq. in.
Crown	4275 lb. per sq. in.

Following is a table showing relation of compressive strength of cores to slump and also the average age and number of specimens tested:

Slump in Inches	Number of Cores Tested	Average Age in Days	Compressive Strength, lb. per sq. in.
0 – ¾	1	270	4275
1 –1½	20	189	4250
1¾–2¼	28	190	4292
2½–3	43	206	4839
3¼–3¾	16	175	5075
4 –4½	44	166	4872
4¾–5¼	28	208	5011
5½–6	16	234	5241
6¼–6¾	13	265	5385
7 –7½	2	396	5452

Cores that were too badly honeycombed for testing were not included in the table. Scarcely any honeycombing was observed in the range of 1¾–2¼-in. slumps and no honeycombing appeared in concretes made with higher slumps.

From an examination of the table it appears that the concrete with the higher slumps gave the greatest strength even after allowances due to differences in age are made. This is probably due to the greater workability of the higher slumps and the better placement secured under

tunnel conditions. This data differs so widely with the data obtained from the test cylinders (Fig. 13) that it may be well to call attention to the fact that test cylinders may not represent job conditions where working with the drier concretes. This is especially true in view of the fact that test cylinder concrete is rammed some 400 strokes per cu. ft. of concrete, a very unlikely procedure with job concrete.

This important question arises: When high early strengths are not essential but the best concrete ultimately is required, should not wetter mixes be used than now advocated, provided the mixes are not so wet as to induce segregation?

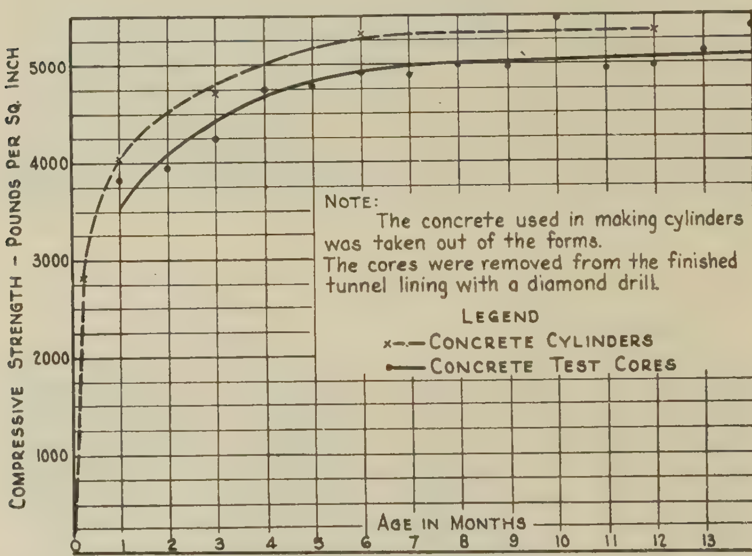

FIG. 18. COMPARISON OF COMPRESSIVE STRENGTHS OF CONCRETE CYLINDERS AND TEST CORES.

Fig. 18 shows a comparison of the strength of the concrete cores at ages up to 12 months as compared with the test cylinders. It can be seen that the cores are somewhat lower in strength than the strengths as reported by the test cylinders. This is probably entirely due to the lower crown strengths because the crown strengths as reported by the cylinders were always the highest whereas it can be seen from the foregoing table that the crown strengths as reported by the cores were somewhat less.

Several cores were taken through the joints. A compressive test of one core through a sound joint showed a strength of 4456 lb. per sq. in. for concrete about 3 months old. The corresponding 28-day cylinder

had a compressive strength of 4430 lb. per sq. in. Cores at the springing line taken through hair-cracked joints showed the opening was mort pronounced at the face of the tunnel lining.

In no case was grout discovered below the upper quarter point and usually the grout did not even extend this far. The grout was in all cases well bonded to the concrete and was very hard. Usually the grout penetrated to some extent into the concrete, in one case as much as 5 in. The average thickness of grout above the concrete was about 1½ in. and in every case completely filled the space between the concrete and the surrounding soil.

The thickness of lining was found to run well over the contract requirements and on the whole checked the inspectors' measurements closely. The percentage of excess concrete placed is shown by the following:

Section 1	6.5 per cent
Section 2	12.1 per cent
Section 3	17.7 per cent
Section 4	12.0 per cent
Section 6	12.3 per cent

Such success as has been obtained in securing a good grade of concrete in the water supply tunnels is believed to be due to:

(1) An appreciation of concrete control methods and incorporating them in the specifications, and

(2) To thoroughgoing inspection with careful attention to every detail.

The writer wishes to thank the various members of this department who have been connected with the work for their thoroughgoing co-operation without which these results could not have been obtained.

Mr. Rockwood. E. F. Rockwood—I would like to ask Mr. Lenhardt if there were no forms around the outside, or whether the earth was strong enough to serve as a form?

Mr. Lenhardt. L. G. Lenhardt—Where the earth was not strong enough to serve as a form, timbering of various types was used. For the most part we got by the work with comparatively little timbering; in other words, twelve to fifteen planks.

Mr. Munsell. A. W. Munsell—Did you use hammers on the forms where the cores showed higher strength than the cylinder concrete? The idea is that the hammers may have impacted or consolidated the concrete and driven out the entrained air and water and made a change in the water-cement ratio.

Mr. Lenhardt. L. G. Lenhardt—I do not believe that is the whole story. We took some cores at points where hammers had been used and where the slumps were practically the same as at the crown; practically the same results were obtained as at the crown where hammers were not used.

Mr. Munsell. A. W. Munsell—May I ask how long you kept the hammers on these plates?

Mr. Lenhardt. L. G. Lenhardt—That is something we experimented with considerably and are not done experimenting with yet. Each plate was rammed probably about ten seconds. We found that the hammers should not be used too quickly; in other words, the air hammers should not be used as soon as the concrete was placed. We tried to keep about two plates below the concrete.

Mr. Giles. R. T. Giles* (*By Letter*).—Mr. Lenhardt has brought out many interesting points in his paper and has displayed an understanding of the subject too seldom found on actual construction work. One point in particular is of especial interest to the writer, and since he has investigated in a small way the point in question, it is possible that the results of that investigation may be of interest.

Reference is made to the fact that the *compressive strength of the cast cylinders, with regards to slump, were directly opposite to the compressive strength of the cores drilled from the placed concrete and therefore misleading.* If concrete which can be commercially placed without honeycomb is accepted as workable, then it would appear from information given that *it is possible to have a workable concrete with too little mixing water to produce the best results.*

It was noted several years ago from tests on cores from concrete roads that the drier concrete, although workable for the purpose used, did

* Engineer, Atlas Lumnite Cement Company.

not necessarily give the best results. It was felt that for drier mixes, due to the excess rodding given to cast cylinders, the results from the previously cast specimens were misleading. If the water used in the drier mix was insufficient to overcome the bulking action of the concrete, similar to that of sands, *this bulking action would not be overcome or reduced in*

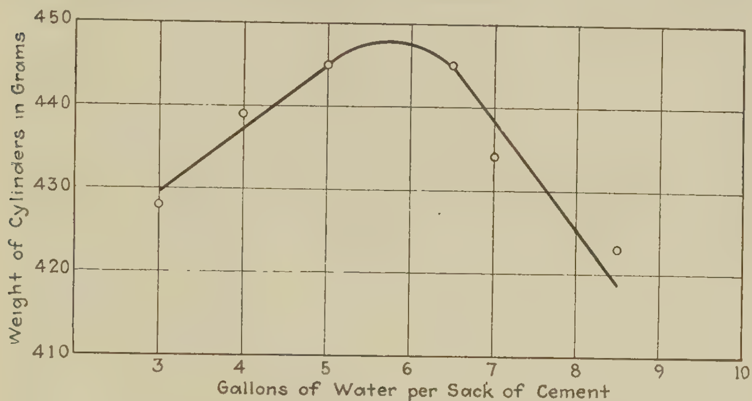

FIG. 1.—RELATION OF WEIGHT OF 2 x 4 MORTAR CYLINDERS TO GALLONS OF WATER PER SACK OF CEMENT.
Artificially graded sand using 25 per cent cement. Age at test 28 days.

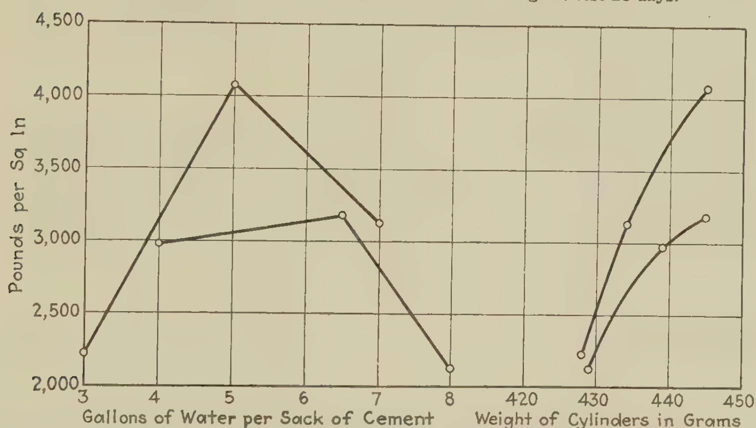

FIG. 2.—RELATION OF 28-DAY MORTAR COMPRESSIVE STRENGTH TO GALLONS OF WATER PER SACK OF CEMENT AND TO WEIGHT OF CYLINDERS.
Artificially graded sands using 25 per cent cement.

the placed concrete which receives a small amount of rodding, but would to a very large extent be overcome in the cast cylinders which receive excessive rodding. It was felt that this was possibly the explanation for the condition. If this surmise is correct, then it would appear that *the amount of mixing water which would give the heaviest concrete per unit volume was the*

best and that this might be a quick means of determining the basic water content for any given mix, since it is well known that excess water occupies space in the concrete and reduces the weight as well as the other qualities of concrete.

When the method of making a comparison in an investigation was discussed, it was felt that, due to the justifiable prejudice in favor of standard methods of tests, any reduction in the number of roddings given a cylinder would possibly reduce, if not nullify, the value of an investigation in the eyes of those who advocated standard methods. It was therefore decided to make an investigation on 2 x 4-in. mortar cylinders. To eliminate the possibility of questionable procedure in favor of the wetter mixes, dry mixes were rodded until no more material could be forced into the cylinder, while the wetter mixes were rodded the usual number of times. While it may be possible to overcome by rodding to a very large extent, the bulking of concrete caused by too little mixing water, it is practically impossible to overcome the bulking of mortar by rodding. For this reason it was felt that 2 x 4-in. mortar cylinders would illustrate the condition, although it was recognized that the bulking condition with mortars was more aggravated than would be the case with concrete.

Tests were made, using three different water-cement ratios for each set. Twenty, twenty-five, thirty and thirty-five per cent cement was used. Tests were made at 28 days and 6 months, while some of the cylinders were tested wet and some dry. Ten cylinders for each average were made for the strength tests and to determine the weight. In two of the sets the sand was artificially graded, while in the other an average natural sand was used.

Fig. 1 shows the strength of the cylinders plotted against the gallons of water and the weight of the cylinders, and Fig. 2 shows the gallons of water plotted against the weight of the cylinders at 28 days, using 25 per cent cement with the artificially graded sands. The following 6-month results of Series 25-1, Section C, and 28-day results of 26-3 are typical.

SERIES 26-3—28 DAYS—ARTIFICIALLY GRADED SANDS

(Each result is average of 10 cylinders)

Water (gal.)	4	6½	8½
Strength (lb. per sq. in.)	2950	3350	2275
Weight (gr.)	439	445	429

SERIES 25-1—6 MONTHS—NATURAL SAND

(Each result is average of 10 cylinders)

	20 Per Cent Cement			25 Per Cent Cement		
Water (gal.)	3	5	7	3	5	7
Weight (dry)	411	419	421	419	432	439
Strength (dry)	1513	2873	3338	2269	4069	4963
Strength (wet)	1132	2085	2610	1698	3357	3963
	30 Per Cent Cement			35 Per Cent Cement		
Water (gal.)	3	5	7	3	5	7
Weight (dry)	430	448	439	442	457	429
Strength (dry)	3085	6176	5039	4718	7039	5203
Strength (wet)	2131	4777	4300	2960	6000	4311

It is interesting to note that the heaviest cylinders invariably gave the highest strength results while the lightest cylinders did not always give the lowest results. The excessive rodding of the drier mixes probably accounts for this fact. In some cases the excess water produced a heavier concrete than too little water, but in other cases the reverse was true. Another interesting point in this connection is the fact that the yield of the drier concrete must have been greater than the wetter concrete since the weights reported by Mr. Lenhardt showed the wetter concrete to be the heaviest. This information does not check and in fact is just the reverse of the formula so often used, that the sum of the absolute volumes of the materials, including the water, will give the yield of the concrete mix.

The facts brought out by Mr. Lenhardt and the trends of this investigation would seem to indicate that further work along this line might be well worthwhile, and that a definite quick means for determining the most economic amount of mixing water for a given combination of aggregates may be worked out.

Supervision and Inspection of Concrete in Modern Building Construction

By J. M. Bischoff*

During the past few years, building operations in the City of Detroit have been exceedingly active, and it may not be amiss to give a brief synopsis of the precautions taken by the Department of Buildings to safeguard the public by insuring proper construction methods and the use of correctly proportioned concrete in our modern skyscrapers. It is not my intention to enter into the features of concrete making, but merely to outline the system adopted by our department for checking up the quality of the concrete both in the laboratory and in the field.

Of the various types of foundations in use, namely the spread foundation, the mat foundation and the open well type of caisson, where the bearing of such foundation is placed on materials other than rock, we shall consider but one, the caisson type. When caisson foundations are built to varying depths, the laboratory is frequently called upon to make analyses of the quality of the water which seeps in at varying levels; in addition if poisonous gases are encountered their nature must be determined. This latter precaution is particularly important where the caissons are carried to bed rock. In one such instance a considerable volume of hydrogen sulphide gas formed in a pocket directly above the water at the bottom of the well which was supersaturated with the gas and two men lost their lives when lowered into the caisson unaware of the gas pocket. Another necessary precaution is to ascertain the air pressure required to hold back the water which may tend to seep into the shaft. In those ways the lives of the workmen engaged in the operations are protected.

When plans are submitted for approval, the concrete inspector is notified by the Permit and Structural Engineering Bureaus, that a certain permit for a concrete structure has been issued. The inspector then visits the scene of operations, and looks over the materials which are on the ground ready for use.

Samples of the aggregates are next submitted to the laboratory for screen, fineness modulus, and organic matter tests. This is simply an index to the size and grading, which indicates a proper aggregate for use in concrete according to a given specification. If the materials are not up to specification they are at once rejected.

* Commissioner, Buildings and Safety Engineering, Detroit, Mich.

Where a contractor elects to use commercial gravel, the proportions of the aggregates and cement are required to be as stated in the present code. However, where a separate aggregate is used and a proper mixing plant is provided, the proportions of each material entering the concrete mix is governed by the water-cement ratio. This method assures a saving in the amount of cement used as well as the proper amount of water necessary to produce a workable mix.

Another important factor entering into the making of good concrete, and one to which our inspectors pay strict attention, is the time of mixing

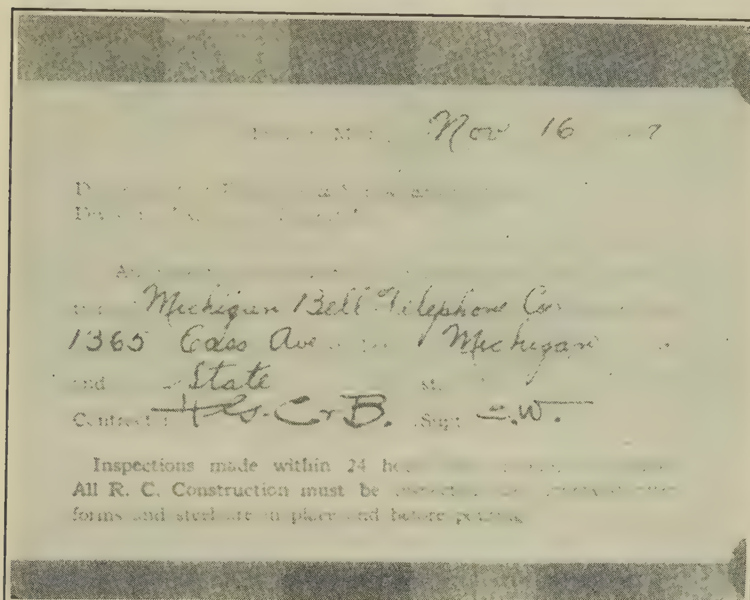

FIG. 1.—INSPECTION REQUEST CARD FURNISHED CONTRACTORS.

and the speed at which the mixer is operated. Best results are obtained when the mixer revolves at 15 to 25 r. p. m.

At the time of the inspector's first visit, he leaves a number of cards (Fig. 1) with the superintendent in charge of the job. When conditions are ready for pouring, the superintendent fills out one of the cards and mails it to the department so that the inspector will get it the following morning. In this way the department is kept informed at all times, just when and where pouring operations are being conducted daily.

Another advantage in this scheme is, that the inspectors can allocate their work so as to cover the maximum territory daily. The average number of inspections made per day is seven, and all jobs whether large or small obtain the same supervision.

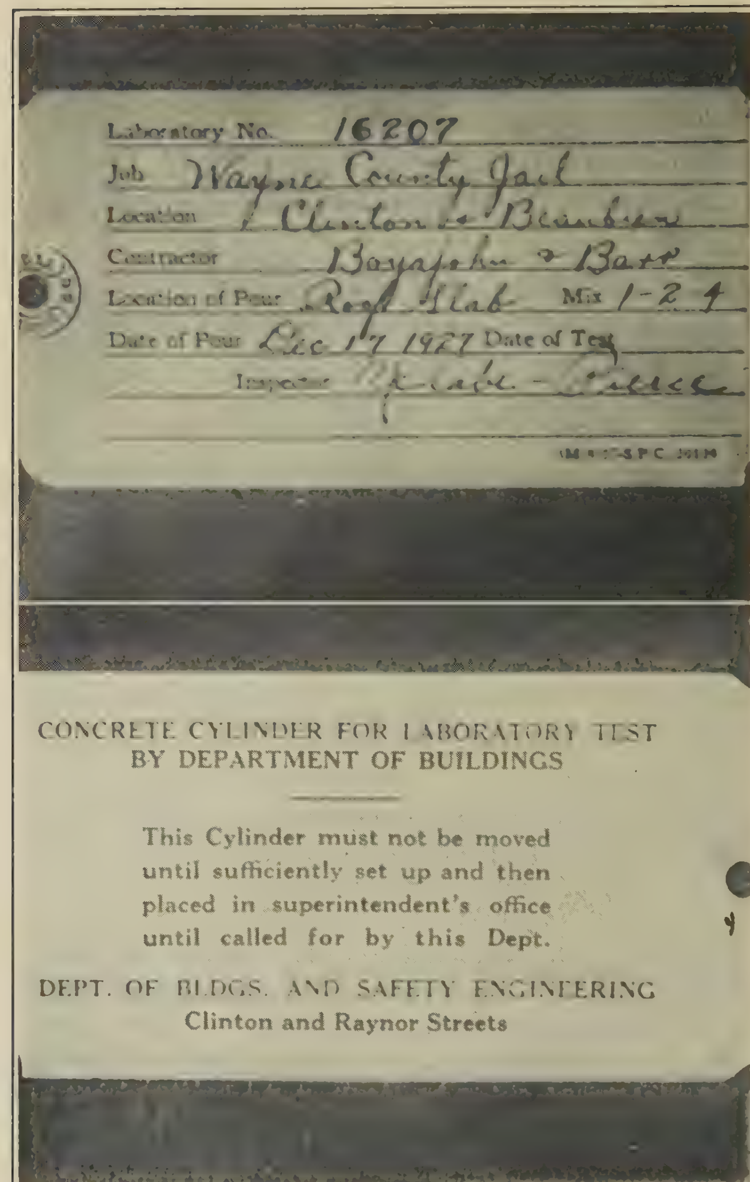

FIG. 2.—FRONT AND BACK OF CYLINDER IDENTIFICATION TAG.

FIG 3—A TEST CYLINDER AS RECEIVED BY THE LABORATORY.

FIG 4.—NUMBERS PAINTED ON TOP AND SIDE OF CYLINDERS TO INSURE PROPER IDENTIFICATION.

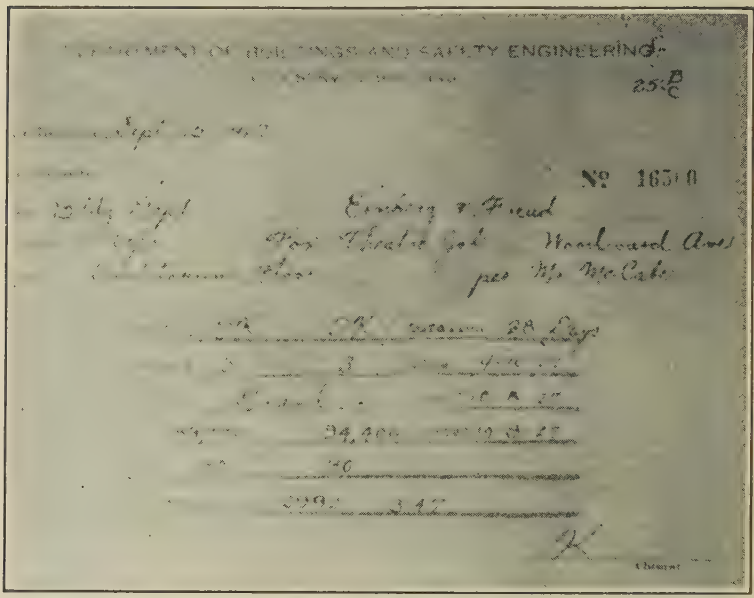
SAFETY ENGINEERING
№ 163
Woodward Ave
per Mr. McCabe
28 Days
94,400

FIG. 5.—LABORATORY MASTER RECORD FOR TEST CYLINDER DATA.

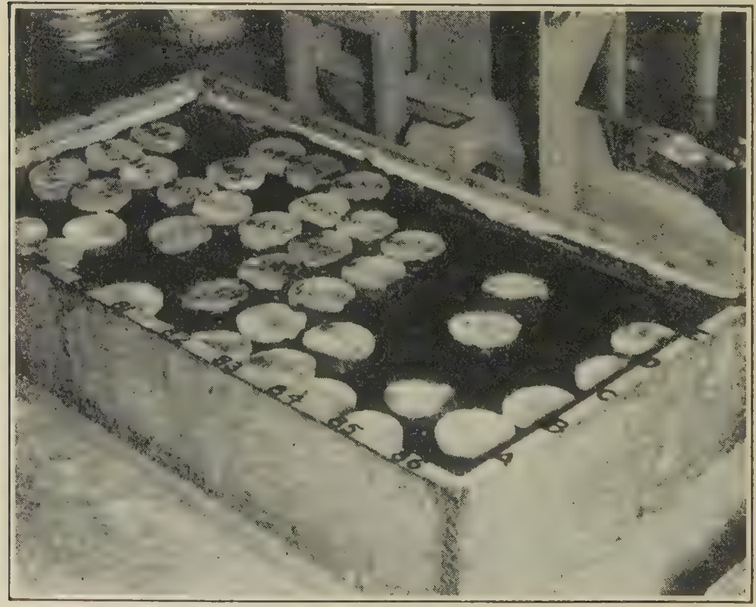

FIG. 6.—CURING VATS SHOWING SYSTEM OF LOCATION NUMBERS FOR THE VARIOUS CYLINDERS.

Molds for test cylinders as furnished by the department are constructed of 5-ply paper, well paraffined inside and outside, and have given good results. Duplicate samples are always taken in accordance with A.S.T.M. specifications, the molds being filled under decking and stored there or in the superintendent's shanty, depending upon conditions,

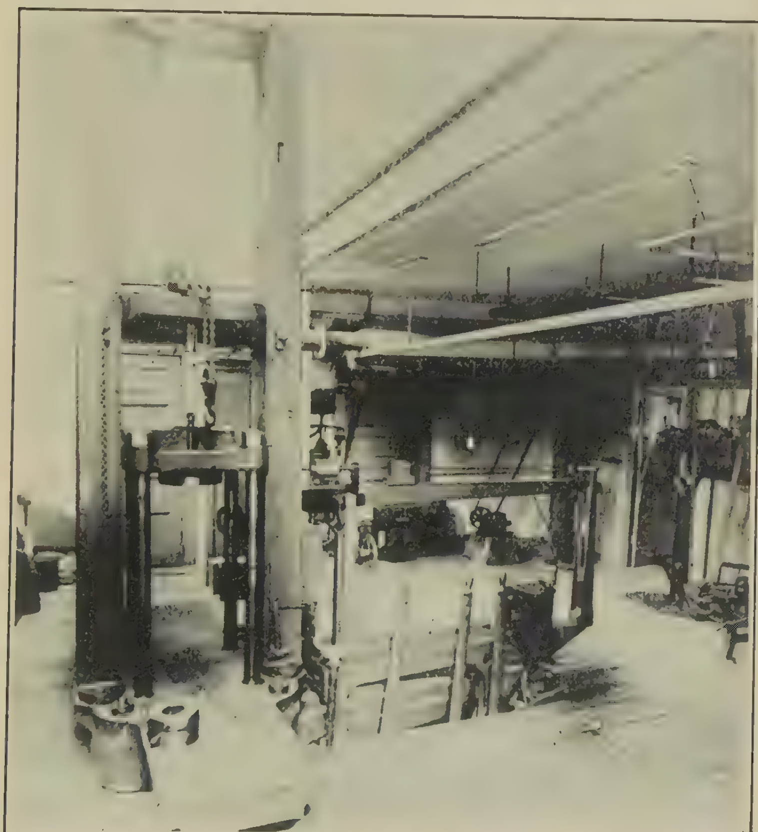

FIG. 7.—TESTING MACHINE IN DETROIT LABORATORY.

until the pick-up wagon arrives. The inspector notifies the laboratory in writing of the locations where pick-ups are to be made.

To identify the cylinder so that a permanent record of all the data may be on hand, tags as shown in Fig. 2 are affixed to each. The tag indicates the job, location, contractor, place from which the tests were taken, mix, date and inspector's name. In addition the signature of the superintendent or his recognized representative is insisted upon to pre-

clude any argument relative to the tests. After the data has been transferred to the laboratory master sheet, these tags are filed away for a reasonable length of time.

The advantage of this arrangement is obvious, because it conserves time and expedites receipt of the cylinders so that they may receive the proper curing necessary. During the busy season about 40 cylinders a day are received from all sections of the city.

In Fig. 3 is shown a cylinder as it is received by the laboratory, while Fig. 4 is a view of a cylinder with the mold removed and indentification numbers painted on. When two cylinders of the same material are received the same identification number is painted on each.

The laboratory master record as shown in Fig. 5 contains all the data which was on the tag made out by the inspector when the cylinder was cast. In the upper right hand corner the number 25 followed by the letters *B* and *C* is a laboratory notation indicating the location of the cylinders in the curing vats. This is cylinder No. 16,500; the numbers run consecutively and the forms are made up in books of 100 sheets for handiness. The meaning of the other data on the sheet is obvious.

Another record kept by the laboratory, and one which has proved its worth on many occasions, is the cylinder record book with which, given the approximate date, the job and contractor's name, the master records of all the tests from the particular job in question can be located and produced within 30 minutes.

The cylinders are next transferred to the curing vats (Fig. 6). Along one side is shown a series of letters, while on the front is a series of numbers. On the top of the cylinders the painted numerals are visible, so that with this system any cylinder can be readily located. An important point in the curing is the temperature range of the water in the vats. This remains reasonably constant, ranging from 65 deg. F. in winter to 80 deg. F. in summer.

At the end of the curing period, the cylinders are removed from the vats, usually early in the afternoon, and capped with plaster of paris. They are subjected to compression early the following morning, thus not being allowed to dry out.

Our testing machine has a capacity of 300,000 lb. and embodies several special features, the principal of which is a series of 12 speeds, ranging from .017 to 6.5 in. rise or fall per minute. The cylinders are subjected to load at the rate of .017 in. per min. per ft. of length.

Fig. 8, a cylinder, is shown in position for testing. On top of the steel plate suspended by an arm from the top of the movable head a universal joint is placed, and by proper adjustment of the joint preparatory to testing the cylinder, a great saving of time is effected.

In order to identify types of cylinder failures, a series of typical breaks were photographed and combined as in Fig. 9. Copies of this illustration were sent to all interested parties, so that when a report states that a No. 3 break has occurred, a clear idea of the appearance of the cylinder is obtained by reference to the photo.

FIG. 8.—CYLINDER IN MACHINE READY TO BE TESTED.
An universal joint on top of the plate suspended by an arm from the movable head assists in rapid adjustment of the cylinder in the machine.

FIG. 9.—TYPICAL TYPES OF CYLINDER FAILURES.

With the tests finished, the master copy (Fig. 5), which at the time of filing had several blank spaces, may be completed. First, as regards condition, we record any remarks relative to the ends or general condition of the cylinder. Next there is recorded the type of breakage, followed by the type of aggregate used. Then we record the compression load, the area, and finally the result in pounds per square inch. The record is now

CAISSONS

CONTRACTOR	NO OF TESTS	LOW	HIGH	AVERAGE
1	101	2452	6570	4408
2	137	2579	7062	4966
3	187	1896	6407	4103
4	328	1767	6163	3363
5	24	2108	6227	3645
6	24	2599	5725	4359

GRAND AVERAGE JANUARY 1st to DEC. 31st, 1927 3985 Lbs per Sq. In.
GRAND AVERAGE JANUARY 1st to DEC. 31st, 1926 3837 Lbs per Sq. In.

CAISSONS

CLASSIFICATION ACCORDING TO STRENGTH
POUNDS PER SQUARE INCH

	Year 1926	Year 1927
1000 - 1500	0	0
1500 - 2000	2	9
2000 - 2500	4	63
2500 - 3000	14	96
3000 - 3500	16	120
3500 - 4000	16	131
4000 - 4500	10	136
4500 - 5000	8	96
5000 - 5500	7	84
5500 - 6000	0	34
6000 - 6500	1	25
6500 - 7000	0	6
7000 - 7500	0	1
TOTALS	78	801

FIG. 10.—TABULATION OF DATA SHOWING QUALITY OF CONCRETE PLACED IN CAISSONS IN DETROIT.

complete and is turned over to have the requisite number of copies made to be distributed to the proper parties.

When cylinders test below 1800 lb. per sq. in., floor loads are required to be made under the supervision of the inspector. At this time also, photographs are taken for permanent record purposes. On the outcome of this load test the concrete is either passed or ordered out for replacement.

Another record which has been inaugurated by the laboratory consists of a 4-months cumulative tabulation of data as shown in Fig. 10. It is mailed out to the architects, contractors, engineers and others interested in the concrete situation in this city. This tabulation has proved valuable, in that it shows just what qualities of material are being produced and used as Slab, Footing, Column, and Caisson concrete. The tabulation is prefaced by a letter of transmittal so that any instructions or other remarks of this department can be brought directly to the notice of the parties vitally interested in concrete production. No effort is made to distinguish between a rich or an ordinary mix. All tests are enumerated under one of the above headings. In Fig. 10 such a tabulation on caisson concrete is illustrated. The first column of the upper table headed "Contractor" is in code form, and this code is known only to the chief chemist and myself. The other columns show the number of tests and the high, low and average values of all tests. In the lower table, "Classification According to Strength" is a résumé of the situation from which can be determined improvements or retrogressions in any one year compared with any other.

While I have endeavored to touch briefly on the inspection of concrete only, there are many other phases to which the inspector must give careful consideration. Chief among them are plumb forms; the proper placing of the steel; the length of time required for curing before removal of forms; the proper placing of shoring; and the proper cut off, when pouring is stopped for the day. Also in hot or freezing weather he must see that reasonable precautions are taken to protect the concrete.

One more important point is that the architect should satisfy himself that a competent superintendent is in charge of operations. In dealing with concrete, he has to confine himself to a product which has to be made in the field which is a more difficult task than that encountered in a structural steel building, which is fabricated at a shop by skilled workmen. The latter is under the supervision of a competent superintendent and it seems reasonable that concrete be given the same careful supervision.

Is a Specific Ferro-Concrete Style Evolving?

By Francis S. Onderdonk, Jr.*

One might claim there exist as many different varieties of "modern" architectural style as there exist modern architects. Likewise concrete has been treated in so many different ways that it might appear doubtful whether the term "ferro-concrete style" is justified.

A group of deplorable architects have used the limitless possibilities of "liquid stone" to create monstrosities; the Goethaneum Temple in Dornach, Switzerland, might be considered the greatest aberration of this kind.

Most architects believe that wooden forms alone can be used in creating concrete structures, and hence a very primitive "wood-centering style" has come to be considered typical of concrete by many. Yet self-centering metal reinforcement (the German "Bohm System," composition forms, curved metal forms and the possibility of modeling concrete enable curved and intricate shapes which are truly characteristic of "liquid stone." Hundreds of examples which feature this *plastic* quality of concrete exist in America and Europe and it is these which justify talking of a ferro-concrete style—for wood, rolled steel, stone and brick which have dominated all architectural styles in the past have lent their *angular* shapes to the architecture they helped create. Never before was it possible to pour stone and give it a spine of steel to carry tension. (Figs. 1, 2 and 3.)

In looking over papers dealing with concrete and architecture in the A. C. I. *Proceedings* of previous years one seems to hear an apologetic tone prevail; as if concrete deserved the Cinderella rôle assigned to it by most architects but was not quite as ugly as generally believed.

This paper takes a very different viewpoint: Ferro-concrete not only has architectural merit along with the other recognized materials but is destined to be the material that dominates the modern style; the excellent qualities of liquid stone make it *the* architectural material of our age.

Two important characteristics of the ferro-concrete style are discernible: *pictorial concrete tracery* which will revolutionize the façade-treatment, and the *parabolic arch* which will govern the outline of windows, doors and vaulted halls.

The French brothers Perret created the all-concrete churches in LeRaincy and Montmagny (Fig. 4) in which the entire walls consist of

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concrete grilles composed of pierced circles, squares and crosses. Frank Lloyd Wright has built several residences in California with "textile block slabs," some of which have ornamental piercings. These piercings enliven the façade with their clear-cut shadows, and on the interior cover the wall with an all-over light and shade pattern. The effect is so

FIG. 1.—SCHOOL OF ARTS AND CRAFTS, HAMBURG, GERMANY.
The beams of the second floor are typical of the "wood centering style" whereas the beams of the ceiling express the plastic quality of "liquid stone" and are characteristic of "ferro-concrete style."—Prof. F. Schuhmacher, architect.

novel that only pictures can fully convey the impression. (Fig. 5.) A student of the history of architecture realizes that since the ancient days in which the first true arch was constructed nothing more revolutionary was inaugurated in the field of architecture. Transitional examples of concrete tracery are St. Jean de Montmartre, G. Atterbury's Forest Hills

Gardens tower and Wieleman's register grille. An Italian example is the roof-balustrade shown in Fig. 6, which consists of flowers growing out of a reinforced horizontal stem; this tracery crowns the building most attractively, especially with a blue sky seen through its interstices. The reinforced concrete balustrade combines safety and beauty, for unity

FIG. 2.—WATER TOWER OF THE MAGGI FACTORY, SINGEN (HOHENTWEIL), GERMANY.

of material is one of the essentials of good architecture; a wooden or iron railing on a concrete building will never be as perfect as a concrete one.

The above examples as well as Fig. 7 show the technical possibility of creating figural concrete tracery. Every desired design and scene can

be created by the silhouette effect of ferro-concrete backed by the curvilinear window holes, thus giving by day a dark and at night a lit-up background to these stone silhouettes. They will be very monumental, as they need no bas-relief to be effective.

FIG. 3.—HOUSE AT REAR OF RHEIMS CATHEDRAL, FRANCE.
Concrete frame with brick and stone filling.—Prof. L. Margotin-Thierot, architect.

The concrete framework of piers and girders which form the skeleton of the façade will become veritable frames for pictures and ornaments wrought in concrete tracery. Gothic tracery bars were limited in thinness; reinforcement by aluminum wires will enable the creation of very

thin rods in concrete tracery. Gothic tracery partly served to strengthen the window panes; in concrete tracery this will only be of secondary importance, since the new tracery will act as bracing for the bearing members. The reinforcement will tie concrete tracery and structural frame into *one*; will make the entire wall a rigid unit pierced by holes—holes that tell a story.

FIG. 4.—THE ALL CONCRETE CHURCH OF ST. THERESE, MONTMAGNY, FRANCE.

A. & G. Perret, architects.

The modern architect must take the psychology of our present day life into account. Hundreds of impressions such as those created by billboards, electric signs and traffic signals compete constantly for the attention of the pedestrian and automobilist; motion pictures accustom his nerves to strong effects and striking contrasts. The traditional ornament and bas-relief sculpture cast too pale shadows to attract the man in the street, especially in our temperate zone and farther north where there is so little sunshine. Ruskin's advice was never more needed than today "The power of architecture may be said to depend on the quantity

(whether measured in space or intenseness) of its shadow. . . . And among the first habits that a young architect should learn, is that of thinking in shadow, not looking at a design in its miserable liny skeleton . . . let him cut out the shadows, as men dig wells in unwatered plains; and lead along the lights as a founder does his hot metal; . . . his paper lines and proportions are of no value; all that he has to do must be done

FIG. 5.—THE FIRST TEXTILE-BLOCK-SLAB HOUSE AT PASADENA, CALIF.
A combination of concrete walls and block-slab.—Frank Lloyd Wright, architect.

by spaces of light and darkness." ("The Seven Lamps of Architecture," 3/VIII.)

Concrete tracery will "cut out the shadows" and provide the high lights. In the temperate zone, houses need so much window area to admit sufficient light that, in order to provide some restful wall space, the architect is prompted to leave the remaining areas undecorated, and only use the window hole with its deep shadow and clear outline as a motive. As the French architect M. A. Lurçat writes: "Today the prac-

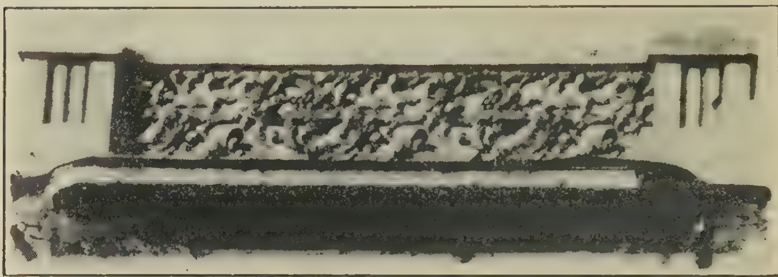

FIG. 6.—CONCRETE TRACERY BALUSTRADE OF A RESIDENCE ON VIA DELL'USINA, GORIZIA, ITALY.

FIG. 7.—BELGIAN PAVILION AT THE 1925 PARIS EXPOSITION OF DECORATIVE ARTS.

The pierced frieze, though executed in non-permanent materials, conveys the impression which will be created by "pictorial" concrete tracery.

tical details of a building, such as windows, with scale and proportion in the balance of solids and voids, have become its cardinal esthetic values."

The customary rows of rectangular windows are monotonous and hence the apertures shown in Figs. 8 and 9. Variegated outlines of the apertures—concrete tracery—is the solution.

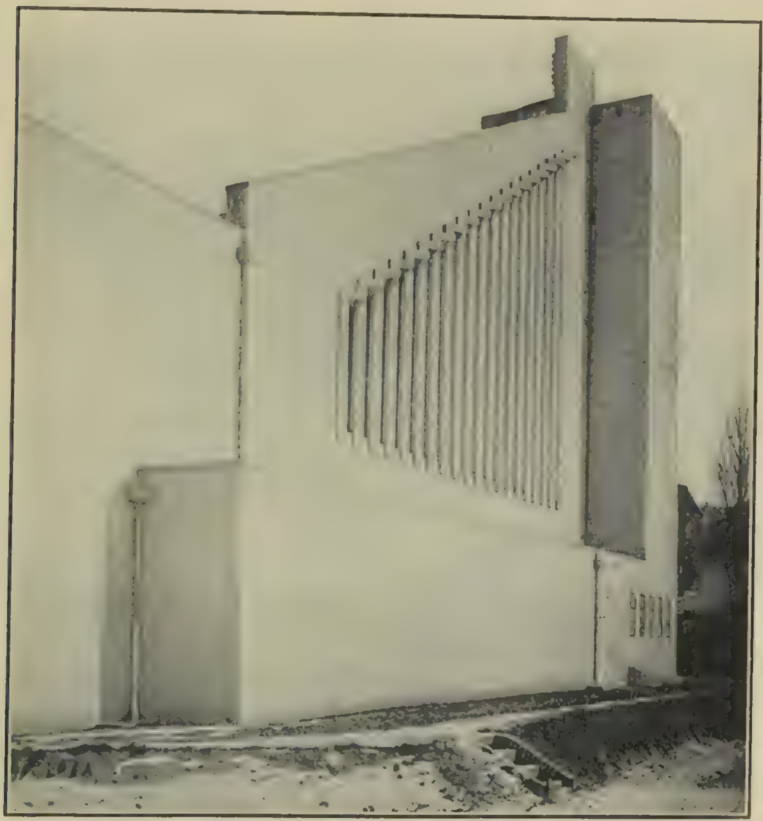

FIG. 8.—SIDE ELEVATION OF THEOSOPHICAL SOCIETY BUILDING IN AMSTERDAM.
I. A. Brinkman and L. C. Van Der Vlugt, architects.

As concrete adopts any form into which it is poured, it is no longer natural or necessary to have vertical or straight contours for wall openings. Steel beams, bricks and wood are straight elements and therefore it has been natural to have windows and doors as well as other parts of a design rectangular, since the introduction of curves necessitated extra cost. In concrete design, curvilinear outlines must become the usual ones, as they are more beautiful.

The wall of the medieval castle was support as well as protection against weather and attack; the small window openings served for light, ventilation and shooting. The wall of the modern frame building consists of a supporting skeleton and protecting panels. In concrete buildings the panel can be further differentiated as an outside shell which bars wind and rain, and an insulating inner shell which, being porous, tends to keep out heat and cold and also is nailable. These curtain walls resemble

FIG. 9.—INTERIOR OF THEOSOPHICAL SOCIETY BUILDING IN AMSTERDAM.

in their constructive function the tympanum of the classical temple and the metopes of Doric friezes; we therefore would be following tradition in covering our concrete wall panels with pictorial tracery. These panels are again divided into concrete strips which serve as bracing, and into voids for lighting and ventilation. The final differentiation will consist in separating these last two functions: built-in window panes such as those in the Veszprem Theatre, Hungary (Fig. 10), which has ventilation

openings in the roof, point in this direction, as well as do big windows with inserted ventilator panes. The light openings can have any desired outline; one pane which can be easily opened will insure ventilation.

As Goethe pointed out, evolution follows a spiral curve, returning after a cycle to the starting point but on a higher level, and so concrete

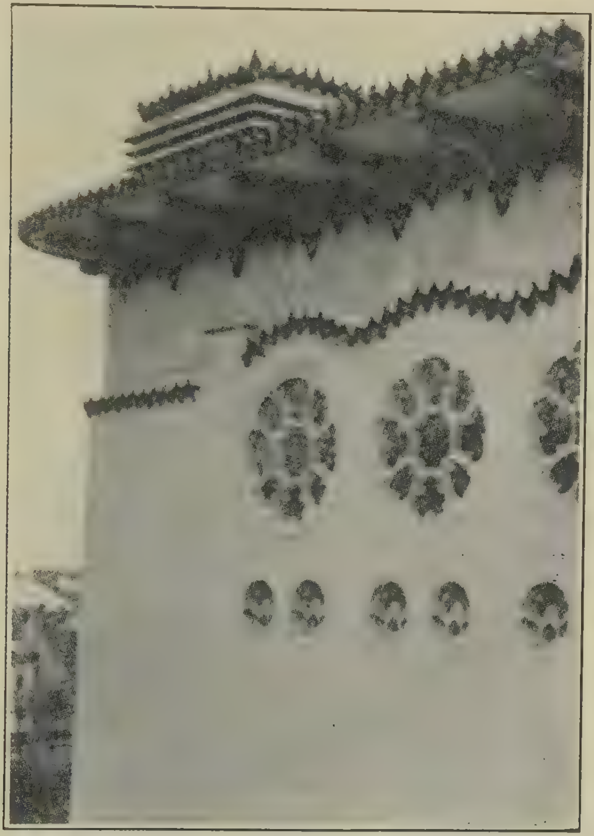

FIG. 10.—BUILT-IN WINDOW PANES IN VESZPREM THEATRE, HUNGARY.

Medgyaszoy, architect.

tracery harks back to the Egyptian tradition of scratching pictures on the early mud or plastered walls. Concrete walls have two points in common with the ancient mud walls of Egypt: plasticity before setting and the desirability of avoiding projections—with concrete a matter of economical centering. On the other hand, ferro-concrete is hard and can carry tension so that it can be pierced; this permits the creation of pictorial sil-

houettes which are far more effective than the incised outlines of Egyptian wall friezes.

Concrete tracery will help the architect to fulfill once more his supreme duty—to create poetry and tell stories in stone—and to be heard. Concrete tracery with its black and white, eventually even with

FIG. 11.—CONCRETE TOWER OF CHURCH ST. LOUIS, VILLEMOMBLE, FRANCE.

color, will convey the architect's message, and in a way to compete successfully for attention with the advertisements which are the most conspicuous features of our modern cities. The true artist always has a message and concrete tracery will be an effective medium for proclaiming it. The thoughtful architect will have to agree with John Ruskin in

affirming that the rudest work telling a story or recording a fact is preferable to the richest without meaning.

Architects have often attempted to force three-dimensional beauty on city house façades (which are essentially two-dimensional) by imposing "orders" on them; but as long as there is no space between colonnade and main wall, the deep shadows are lacking and exactly the opposite effect of the one aimed at is achieved. The zoning laws which have created the stepped-back façade for skyscrapers provide one solution. Concrete tracery will give a second one: the façade is candidly declared a plane which is treated like a *lace curtain.* The many interstices of the tracery show a sufficient amount of the space behind the wall, dark by

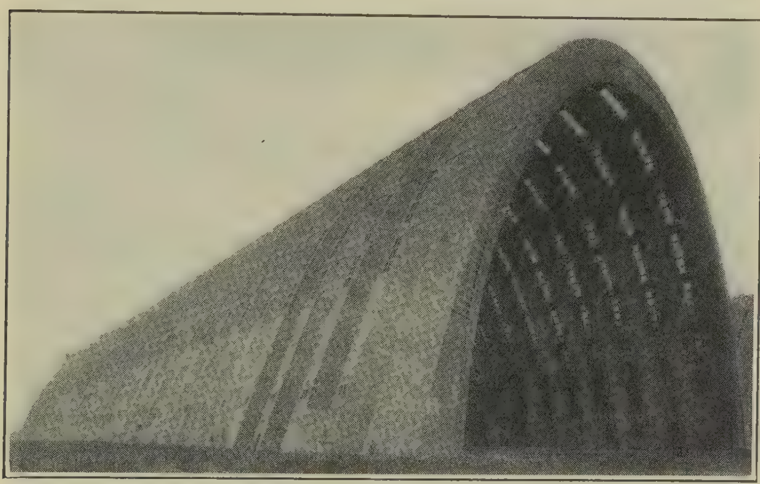

FIG. 12.—AIRSHIP HANGAR AT ORLY, FRANCE.
E. Freyssenet, engineer.

day, light at night, to give the three-dimensional impression of the house, of which the façade is professedly only one plane. Instead of a portico degenerated into pilasters, the concrete colonnade is left in front and the intercolumnar spaces filled by grille work. The house itself—a dark or light space—remains behind this concrete lace curtain and the conquering of space is expressed, especially when a far-projecting cornice and bold balconies are used as finishing touches to the design.

With concrete tracery the fenestration can attain various degrees of ornamentation. The total glass area can be adjusted to meet every demand. The concrete "picture frame" of piers and sills enclosing glass but no concrete "picture" is the one extreme as used for factory buildings; perforated "textile blocks" as designed by Frank Lloyd Wright represent the other.

In monumental public buildings the walls of the large halls would depict historical scenes or a symbolic group of figures. Concrete tracery would be far more effective than frescos wedged between rectangular windows. In case bas-relief is desired in addition to the silhouette effect of the tracery, the concrete could be chiseled before it is quite hard or glue molds could be inserted in the main centering. The inside wall surface, being so much nearer to view, would demand colored or mosaic covered concrete as evolved by J. J. Earley, and possibly stained glass in the voids, as the figures would appear too large and be in shade. Classical art would furnish inspiration for the archaic rigor of the outlines of

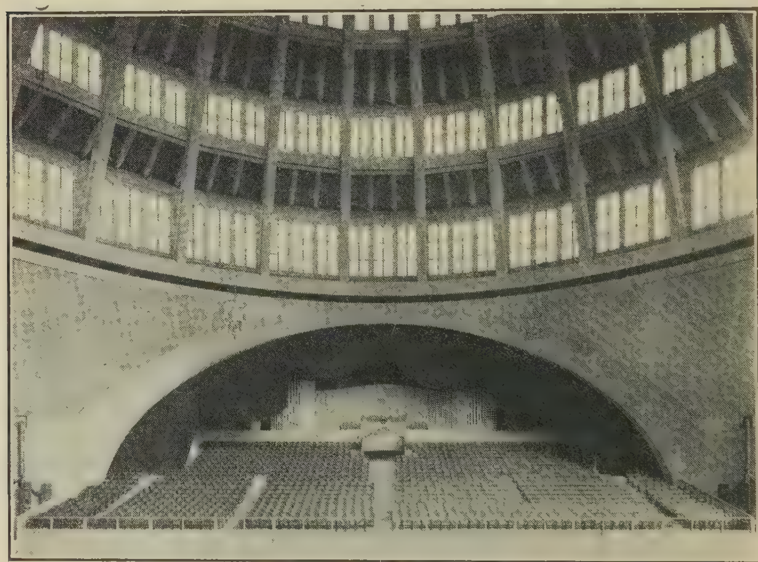

FIG. 13.—INTERIOR OF CENTENARY HALL, BRESLAU, GERMANY.
Baurat Berg, architect.

concrete tracery which would be essential in making it clearly understood when viewed from the distance. Our modern poster art shows the range of effects which can be achieved by simple contours and areas of uniform tone set off by darker ones.

The shadow intensity would be variable according to whether the window panes are set nearer the outer or the inner wall surface. The voids could vary concerning the light they admit: stained glass, glass blocks as used in Europe, or ordinary glass of varying thickness and fluorescence could be used. The concrete itself could be subjected to various kinds of surface treatment and by combining all the mentioned possibilities an almost unlimited number of ornamental effects presents itself.

Concrete tracery can be formed by placing thin strips of metal or inserts of other material between the centering shells. The glass panes might be placed on rebates in the concrete ribs as was done in the Veszprem Theatre or set like ordinary window panes in lead frames. Ventilation panes would be either pivoted, hinged or sliding.

The work of a sculptor can be reduced to a minimum by an ingen-

FIG. 14.—ROYAL HORTICULTURAL SOCIETY HALL, LONDON.
Easton & Robertson, architects.

ious form builder capable of blocking out or of bounding a statue by the flat planes which are already his stock in trade.

The French artist M. Sarrebezolles demonstrated the effect of sculptured concrete masses in the tower of St. Louis, Villemomble, near Paris (Fig. 11). Ordinary concrete from which the large aggregate had been removed was cast in wooden forms and permitted chiseling on the following day. Twenty figures in all were arranged in two tiers, each more

than twenty-one feet high. There are eight small figures and a number of smaller heads to enliven the mass. The whole stands as a symbol of the church, with the great pope Gregory as a central figure. That M. Sarrebezolles had to do as many as four heads in three hours gives an idea of the required speed. The great undertaking was finished in three months; the rapidity with which this work was done indicates the reasonable cost. Thus ferro-concrete enables the architect to return to the Gothic tradition of making the building a petrified motion picture that grips the imagination of the spectator.

Next to concrete tracery the parabolic arch seems to be the outstanding characteristic of the ferro-concrete style. The Markethall at Breslau, Germany, the Exposition Hall in Magdeburg, Germany, and the airship hangar at Orly, France (Fig. 12), are some of the halls that have been executed with parabolic arches. Engineering introduced the parabolic arch as the strongest type and thus we find it in many bridges; it is also executed in masonry as in the Engelbrekt Church in Sweden and in various brick buildings in Holland. The parabolic arch is characteristic of "liquid stone" which, in its absolute freedom to adopt any form is well suited to the ever-changing curvature of the parabola. The parabola in turn expresses the monolithic quality of reinforced concrete by merging sides and top in one unbroken curve. Moreover, due to our modern knowledge of statics, the parabolic arch is recognized as the most economical one. Geology and astronomy have extended our limits in time and space and this may be the psychological reason why the semi-infinite curve, the parabola, seems to express a sentiment of our age. This concurrence of structural and esthetic reasons indicates that the parabola will more and more become the typical outline for arches and vaults in the ferro-concrete style.

Some architects still think that our generation can be satisfied to build in the historical styles and that those who attempt to design in a new way are just seeking publicity. Certain so-called "modern" buildings justify this suspicion. On the other hand, it must not be forgotten that shapes and ornaments, just as phrases, when repeated over and over again, utterly fail to impress. If a building is to make an appeal to the public, it must first gain its attention; to do this the architect is justified in using new expressions for old truths. From this viewpoint the parabolic arch and concrete tracery may be helpful innovations. All depends on the spirit and the artistic standards of the designer.

The many possibilities created by reinforced concrete can also be dangerous because they permit architects to indulge in stunts. In the historical styles the small intercolumniation served as unit for rhythm, but the span between two ferro-concrete columns is too big to act as scale. Reinforced concrete has encouraged some "modernists" to create monstrosities; no other architecture exposes the designer's ability as fully as ferro-concrete does; no other material permits such brutality and crudeness. The freaks of a future degenerate period may surpass those of the Baroque, as ferro-concrete gives the architect more liberty.

Even now the shapes given to the new material by some architects remind one of the manners of a parvenu.

To replace the chaos of modernism by a ferro-concrete style, architects would have to agree on what is good and what is bad in design, and herein Count Leo N. Tolstoy's criterion of art could be applied to architecture: The spectator must be infected by the sentiment which the artist experienced while creating.

Mr. Friis. Kristen Friis, of Norway—It is indeed refreshing to hear a lecture like this, and my confidence in concrete has grown. There is one thing I should like to ask Mr. Onderdonk: Is any special precaution taken to protect the surface of these concrete buildings against weather conditions and frost? We have seen all these beautiful buildings and surfaces and I should feel sorry if this work should deteriorate with the years. I should think it must be possible to protect all concrete surfaces, since we have floor protections which give a chemical combined in the surface that will harden it and make it more resistant against weather.

Mr. Onderdonk. F. S. Onderdonk—I must admit that I have come to your convention for the answer to this question. Since returning from Europe, where I spent twenty years, I have often heard here that concrete is all very well for California and Florida but not for states with a northern climate like Michigan. In central Europe there is often frost, and yet one rarely finds complaints concerning cracked surfaces in German publications dealing with architectural concrete. A German architect to whom I mentioned these conflicting experiences expressed the opinion that the matter was entirely a question of workmanship.

Mr. Bauman. E. W. Bauman—Why the term ferro-concrete? Why not reinforced concrete?

Mr. Onderdonk. F. S. Onderdonk—The expression "reinforced concrete" is not as good as ferro-concrete, because it is too long; a term, to become popular, must be as short as possible. The Germans have the advantage of their short *Eisenbeton* and the French the brief term *béton armé*.

Mr. Mauro. F. Mauro—In February, 1907, I went home to Italy, commissioned by the Concrete Steel Company to see what kind of work was being done there. I saw some of the concrete which was so well described by Dr. Onderdonk. The speaker was perfectly right in using the expression ferro-concrete, because historically it is the name originally given to that combination of iron and concrete. At that time I brought back a photograph of a part of a building showing a quality of work which is very interesting and was well illustrated by the speaker; part of its interest is due to its date, 1907. Some of the work was cast in place and then carved by the stone-cutter; in other cases cured blocks were manufactured and set likes tones. I was told that well-done work of this nature costs about fifty per cent of similar work done in stone.

Mr. Onderdonk. F. S. Onderdonk—Pictorial concrete tracery will enable the architect once more to speak in such a way that people will stop and notice

that there are architects. Traditional ornaments with their faint shadows and hazy outline have become insignificant in our day; they are even considered disturbing, as evidenced by the fact that in Berlin a firm specializes in scraping ornaments off facades which are then renovated with a smooth surface. In the ferro-concrete style the wall will consist of a structural concrete frame filled by silhouettes of concrete against glass.

A. S. BROCK—I would like to ask Dr. Onderdonk what he thinks of some of the efforts of the American architects in using concrete? He referred particularly to Frank Lloyd Wright. A large number of other architects in California have used concrete in many ways. I would like him specifically to comment on the plastering of concrete surfaces to simulate stone and on the making of concrete trusses painted so that they will look like wood. **Mr. Brock.**

F. S. ONDERDONK—The ferro-concrete style is at the beginning of a long evolution. I think architects are more and more realizing that to imitate another material in concrete is bad taste. As John Ruskin wrote fifty years ago, beauty stops as soon as deception begins. Concrete has been treated as Cinderella by the majority of architects, but her beauty has been discovered in France, Germany, and in America in California and a few other states. In *The Ferro-Concrete Style*, published by the Architectural Book Publishing Company, New York, there are 400 illustrations of work of the latter character. Mr. Allison, who addressed a former convention of your Institute, and other California architects have done great things in concrete. In Germany, Professor Bohm created a concrete vault by using self-centering reinforcement, a special mesh with clay units at the intersections. This was plastered with concrete stucco and, after it had set this shell acted as centering for the concrete vault. The cupola of the Zoological Building in Cincinnati was poured in a similar manner. When we liberate ourselves from the carpenter, from the wooden form, we approach the real ferro-concrete style. Precast ornamented concrete slabs were used as centering in Austria as long ago as 1908; Frank Lloyd Wright is demonstrating further possibilities of this method with his precast concrete-block slabs. **Mr. Onderdonk.**

WILLIAM H. ADAMS—I would like to ask Dr. Onderdonk whether these interiors of banks and churches are generally plastered or if some effort is made to treat the concrete surface so that it remains concrete? **Mr. Adams.**

F. S. ONDERDONK—I cannot tell you in each individual case. Both methods have been applied. The one drawback ferro-concrete really has for the architect is that it offers so many possibilities that he does not know which one to choose. **Mr. Onderdonk.**

Volumetric Changes in Portland Cement Mortars and Concretes

By Raymond E. Davis and G. E. Troxell*

Introduction

Looking back over the past 25 years, one is impressed with the tremendous volume of research that has taken place in the field of concrete, yet one is forced to conclude how little is known of this material beyond its strength and elasticity and the factors which influence these properties. Still, this perhaps is not surprising when one considers that concrete is a complex material of many properties, most of which vary with time and other conditions.

Of late we are beginning to appreciate that the permanency of a concrete structure is dependent upon more than mere strength and rigidity, and that certain other properties, concerning which we have very little quantitative data, are frequently of even greater importance in problems of design. Of these properties, perhaps the one concerning which the need for adequate information is greatest, yet least is known, is that possessed of portland cement mortars and concretes of changing their volume, not only during the early period of the hardening process, but thereafter, as variations in moisture conditions occur. Just as clays, and to a lesser extent the rocks of our hillsides, shrink when allowed to dry, and swell when moisture is absorbed, so do concretes and mortars undergo similar changes; and, except they be in water or in air of constant humidity for a long period of time, these volumetric changes due to causes other than variations in temperature are continuously in progress, probably throughout the life of the material.

The causes of these volumetric variations are not entirely clear, but perhaps chemical and physical changes within the mass are always involved. The chemist is likely to ascribe this action to shrinking or swelling of the colloids surrounding the cement grains. The physicist may ascribe the volume change to variations of capillary tension within the pore space of the mass. But regardless of the fundamental causes, volume change is certainly a factor to be reckoned with in engineering design, and concrete structures may hardly be regarded as permanent unless the volume changes which are bound to take place with the passage of time may occur without producing excessive stresses. There are numerous examples of the slow failure of concrete which can only be ascribed to this cause, and it seems certain that this property of expanding upon

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the absorption of water and shrinking as moisture is given off is the greatest single factor at work in the disintegration of our concrete structures. So, while in a variety of ways concrete is an ideal material of construction, like the rocks, it does possess the property of variability of volume which, though small, must be quantitatively considered unless our unfortunate experiences are to continue.

With a view to learning more concerning volume changes in portland cement mortars and concretes, a comprehensive series of long-time tests were begun in the Materials Testing Laboratory of the University of California in 1924. During the interval since that time the program of tests has been enlarged until at present more than 400 specimens of a variety of concretes and mortars stored under a variety of conditions are under observation. A test program has been arranged which contemplates a determination of the effect of variations in the following items:

(*a*) Richness of mix.
(*b*) Gradation of aggregate.
(*c*) Quantity of mixing water.
(*d*) Character of aggregate.
(*e*) Moisture conditions including variations in humidity.
(*f*) Conditions of curing.

Studies are also in progress to determine:

(*a*) The effect of alternating periods of watersoaking and drying.
(*b*) The effect of admixtures.
(*c*) The effect of a surrounding medium.

This paper describes these tests and the conditions surrounding them and gives the results so far obtained. Changes of the nature of those here considered under certain conditions continue to take place at gradually diminishing rates over long periods of time, hence some of the values given in what follows will be somewhat modified in the light of future observations. It is expected that the experiments will be continued for some years to come.

Specimens.—For the major portion of the tests the specimens are 3 x 3 x 40-in. bars in the ends of which are cast ½-in. dia. by 1½-in. rustless steel gage plugs or contact posts. The flat ends of these gage plugs act as surfaces of reference by means of which changes in length of the bars are determined. The plugs are held accurately in position in the molds by the use of metal templets and are grooved near their inner ends so as to eliminate the possibility of longitudinal movement within the concrete bar after the molds have been removed. Most of these specimens have been cast in wooden molds, oiled to make them nonabsorbent, but some of the mortar bars have been cast in dry burned-clay molds lined with muslin. Of these latter, some have been removed from the molds and others have been allowed to remain therein. In all cases, except when noted otherwise, the specimens have been kept moist for the first 48 hr., when the molds have been removed and observations of length have been begun.

There is also included in the program on mortars, a series of tests on brick piers. The cross sectional dimensions of these piers are 13 x 13 in. and the height varies from 4 to 10 ft. Three sides of each pier are protected from the air by waterproof paper, and the fourth side is exposed to the air. At intervals of 20 in. throughout the height of the 10-ft. piers ½-in. dia. gage plugs are inserted in the exposed face, but for the 4-ft. piers the gage-plug bars at intervals of 10 or 20 in. extend through the pier from the exposed face to the face opposite thereto.

Storage Facilities.—At the outset of the tests the need for close regulation of temperature and moisture conditions was manifest. To meet this need special storage rooms have been constructed from time to time until they are now six in number. All of these rooms are electrically heated and the atmosphere is maintained constantly at any desired temperature by sensitive thermostats. For the large rooms circulation of

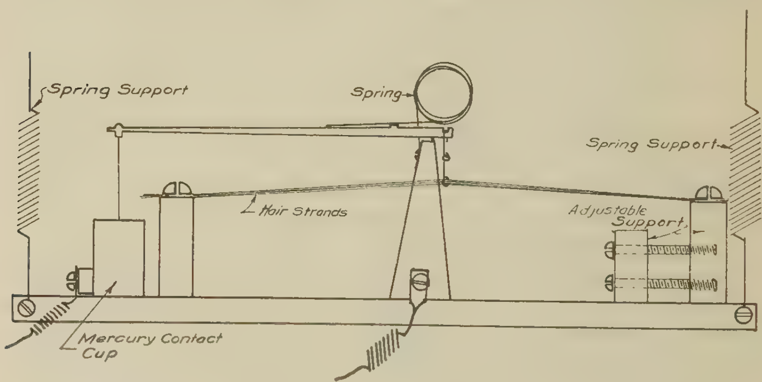

FIG. 1. HYDROSTAT FOR CONTROLLING HUMIDITY.

air by means of blowers is provided in order that the temperature throughout all parts of the rooms shall be constant.

In certain of these storage rooms the humidity is maintained constant by means of hydrostats such as that shown diagrammatically in Fig. 1. The human hair is particularly sensitive to small moisture changes, elongating quickly when the humidity increases and shrinking rapidly when the humidity decreases. The change in the length of the several strands of hair stretched between the two posts of the hydrostat operate the contact lever, the outer end of which closes the electric circuit when it dips into the cup of mercury. Under normal conditions this device serves to control the humidity within 1 per cent.

When the humidity to be maintained is above that of the outside air so that moisture must be supplied, the hydrostat serves to put in operation a vaporizer which continues to function until the humidity is brought up to the desired value. When the conditions are such that water vapor must be removed from the air the hydrostat serves to put in operation a

blower which passes the air through a dehydrator. Fig. 2 shows a corner of Room No. 3 in which the temperature is maintained at 70 deg. F. and the relative humidity is kept at 30 per cent. The dehydrator, shown near the left of the photograph, consists of a tank holding a calcium

FIG. 2. INTERIOR OF ROOM NO. 3.

chloride solution, in the bottom of which is an air diffuser, the tank being surrounded by a water jacket for the purpose of keeping the solution cool and hence increase its efficiency as a dehydrating agent. For nearly all of the tests the temperature in the several rooms has been maintained at 70 deg. F. The moisture conditions have been relative humidities of 45,

50, 70, 75, 80 and 95 per cent, fog by water atomizer at 70 deg. F., damp sand at 70 deg. F., water spray at 70 deg. F., and complete immersion at 70 deg. F.

Apparatus and Methods of Testing.—The general procedure throughout the tests has been to maintain the temperature at a constant value and to subject the specimens to such combinations of moisture conditions as were required by the program. For the bars, observations were begun at the age of two days, except as otherwise noted, and were made periodically thereafter, weights being determined by means of a platform scale reading to hundredths of pounds and changes in length being determined by a special extensometer reading to ten-thousandths of inches. The details of the extensometer are shown in Fig. 3. The device consists of a rigid frame of steel bars to one leg of which is secured a contact post and

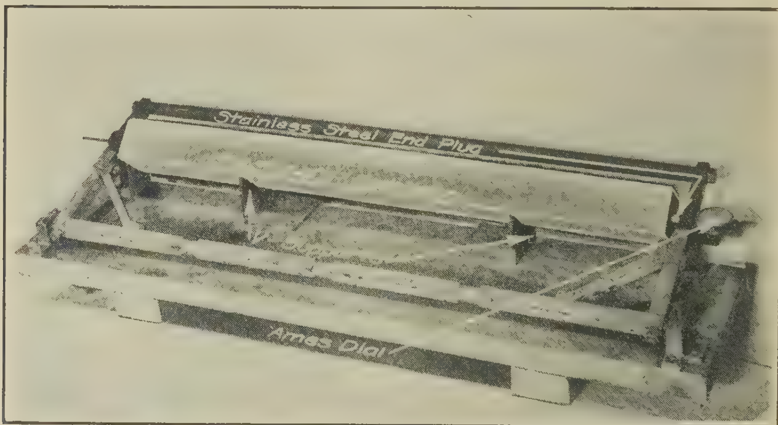

FIG. 3. EXTENSOMETER WITH SPECIMEN IN POSITION FOR OBSERVATION.

to the opposite leg of which is secured an Ames dial directly opposite the contact post. The lower side of the frame rests in bearings attached to the base, about which the frame may be rotated. Fastened to the base are V-shaped supports, and in these supports the specimen is placed, always in the same position. With the specimen in its supports, the extensometer frame is rotated about its bearings until the legs rest on stops in such position that the contact post bears against one end plug of the specimen and the stem of the dial bears against the other end plug. The extensometer is checked by means of a standard steel bar which may be placed in position as are the specimens. Fig. 4 is a corner of one of the storage rooms showing extensometer, standard bar, scales, and specimens. One of the hydrostats is seen near the top of the photograph.

Observations of the brick piers were begun when the piers were 1 day old and were made periodically thereafter, changes in height being

determined by means of strain gages of the fulcrum-plate type similar to the one developed in connection with the arch dam investigation of Engineering Foundation. This type of gage appears to be so superior to any other now in use that its description seems warranted. Fig. 5 shows the details of the 10-in. fulcrum-plate strain gage used on these tests. The two parallel side bars are of nickel steel of channel section having a coefficient of expansion about one-sixth of that of carbon steel. The two side bars are held parallel by two pieces of spring steel (fulcrum plates) at the enlarged sections near the ends of the gage, but the fulcrum plates by flexing allow relative movement of the two bars. The dial is

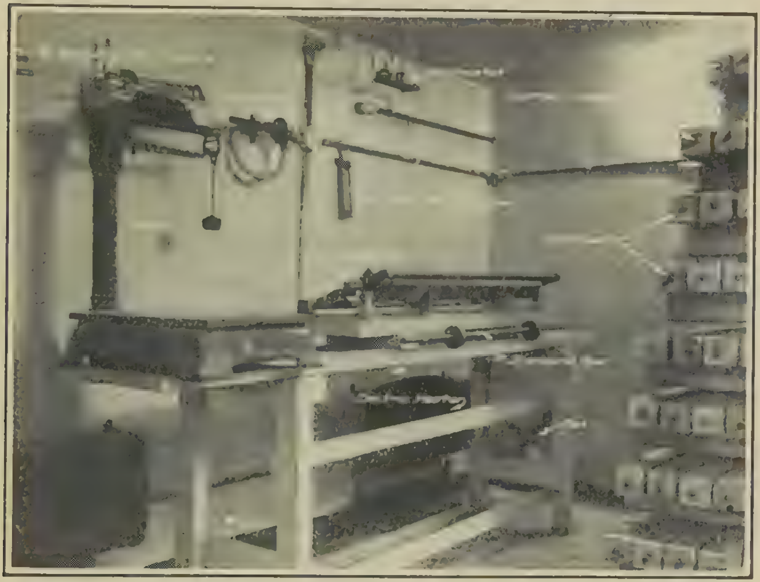

FIG. 4. CORNER OF ROOM NO. 1, SHOWING EXTENSOMETER AND WEIGHING SCALE.

fastened to one bar and the stem of the dial is attached to a lug on the other bar. Hence the dial indicates the relative movement of the two bars. At the extreme end of each bar, beyond the connecting springs, is a steel contact point projecting below the lower surface of the bar. These two points are engaged in the drilled holes of the gage plugs of the specimens, as with other types of strain gages, the gage being held in position by the finger grips located directly over the gage points. The distance between center of dial and lug connecting end of dial stem to opposite leg is made of such length that the thermal expansion in the stem (which is of carbon steel) will offset the expansion in the nickel steel side bars between gage points, hence it is compensating for temperature. The

fulcrum plates eliminate all possibility of lost motion except such as may exist in the dial itself. The fulcrum plates are sufficiently thin so that any normal relative movement of the side bars may be produced by very slight exertion on the part of the operator, hence the points may be properly seated in their gage holes with light pressure. And lastly, the axes of the gage points coincide with the axes of the gage holes when the points are seated.

Scope of Paper.—The tests reported in this paper may be properly divided into two general classes: first, those made to determine the volume changes in mortars such as are used in the laying of brick, stone and terra cotta; and second, those made to determine volume changes in concretes.

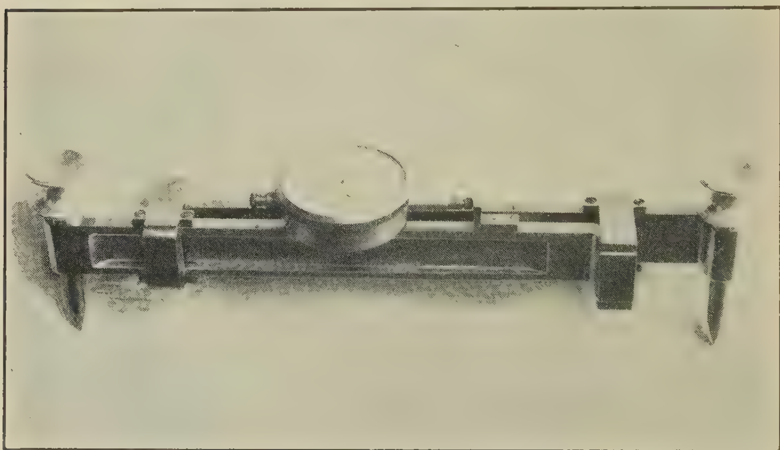

FIG. 5. TEN-INCH FULCRUM-PLATE STRAIN GAGE.

The mortar tests may be divided into three groups: first, those utilizing materials other than portland cement and sand, to produce the plasticity required by the mason for efficient work, such as lime, clay and certain modified cements; second, those in which are incorporated integral waterproofing compounds; and third, plain cement mortars with variations as to richness of mix and water-cement ratio.

The concrete tests may be divided into two series with respect to type of aggregate: those made on concrete for which the aggregate (including sand) is composed entirely of granite; and those made on concrete for which the aggregate is a Coast Range gravel containing a wide variety of sedimentary, metamorphic and igneous rocks including quartz, sandstones, granites, schists, shales, etc.

Those on granite concrete are designed to show the effect of repeated wetting and drying, the behavior when stored continuously in air of

constant humidity, the behavior when continuously under water at constant temperature, and the influence of very fine material upon shrinkage and expansion.

Those on gravel concrete are designed to show the effect of richness of mix, gradation of aggregate, and variations in humidity.

Cement-Lime Mortar Tests, 1926 Series.—The specimens for these tests are mortar bars (cast in non-absorbent molds) and brick piers 10 ft. in height. For all specimens the basic mix was a mortar composed of one volume of portland cement and three volumes of Sacramento River sand. This is a fine sand composed of rounded particles, 94 per cent passing the 28-mesh sieve and 4 per cent passing the 100-mesh sieve. To the basic mix were added various percentages of high calcium hydrated lime, so that for the 5 groups of specimens, the lime content varied from zero

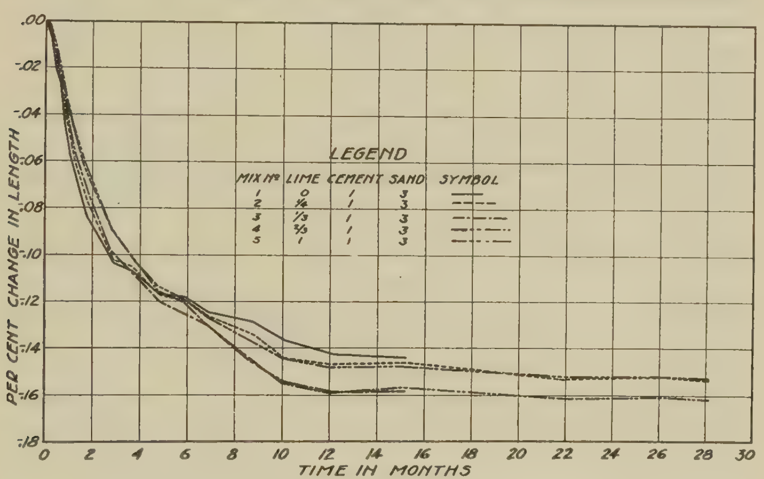

FIG. 6. SHRINKAGE OF CEMENT-LIME MORTAR BARS, 1926 SERIES, AIR STORAGE.

on the one hand up to a volume equal to that of the cement on the other. The consistency of the mortars was such as to produce a slump of 5 or 6 in. Port Costa brick, a medium fine-grained brick (about 15 per cent absorption), were used for the piers, and the average thickness of mortar joint was about ½ in., the total thickness of mortar in the 10-ft. height being about 24 in. The brick were wet with a hose in the usual manner prior to laying, but were by no means saturated when laid.

Both the brick piers and the mortar bars were at first stored in dry air (average temperature 70 deg. F., average relative humidity 55 per cent), but at the age of 7 months part of the bars were stored under water, and at the age of 28 months the piers were subjected to water spray.

Fig. 6 shows the behavior of the five groups of mortar bars stored in air during the 28 months subsequent to casting. Examining the dia-

gram it is seen that the total shrinkage at the end of this period is about 0.16 per cent or nearly 2 in. per 100 ft. It is interesting to note that the shrinkage at the end of two months is roughly one-half that at the end of two years. There appears to be no evidence that the addition of the lime to the basic cement-mortar mix appreciably alters the shrinkage, though the contraction is least for the plain cement mortar. While very little shrinkage took place after the age of one year, from the curves it is evident that shrinkage at a low rate is still in progress at the end of 28 months.

To determine the effect of moisture upon the volume changes, certain of the bars of each of the 5 groups discussed in the preceding paragraph

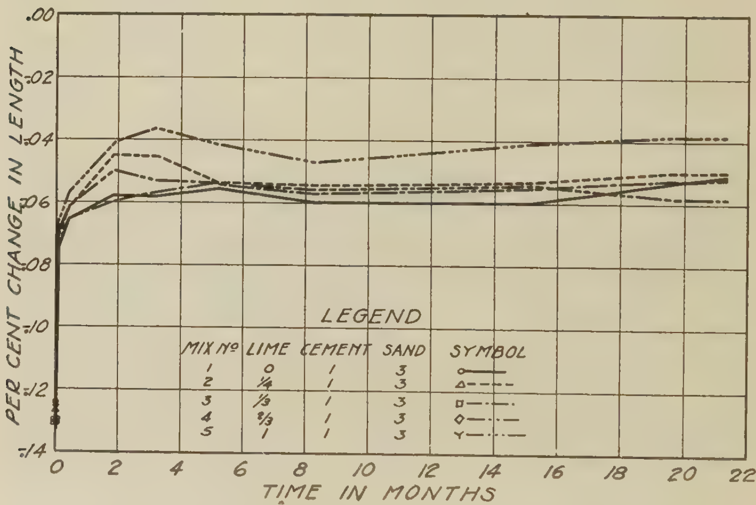

FIG. 7. CHANGE IN LENGTH OF CEMENT-LIME MORTAR BARS, 1926 SERIES, WATER STORAGE FOLLOWING 7 MONTHS AIR STORAGE.

were immersed in water after observations had been in progress for 7 months in air. Fig. 7 shows the results of these tests, the ordinates of the curves being net shrinkages from the beginning of air storage. Within 20 hr. after immersion the bars had recovered about one-half the total shrinkage that had developed during the preceding 7 months in air. There is no evidence of additional recovery after 3 months in water, when there is still a net shrinkage of one-third to one-half that which took place in air prior to immersion. It will be observed that the expansion following immersion is greatest for the mortar with largest lime content and, in general, least for the mortar without lime; but the difference is not large, and there is lack of consistency in this respect when all groups are considered.

Fig. 8 gives corresponding changes in length for the brick piers stored in air for the first 28 months and then in water for 4 months. Consulting the figure it is seen that at first the brick piers without exception elongated quite rapidly, reaching a maximum length after one or two months in air and thereafter shrinking until placed in water spray. With one exception (the plain cement mortar) they reached their original length in 11 to 14 months after construction. The tendency of the piers to expand during the early period of air storage is quite in contrast with the behavior of the bars similarly stored, yet perhaps upon careful thought this might not be unexpected and a possible reason for the difference may be explained. It seems probable that the excess moisture of the mortar was

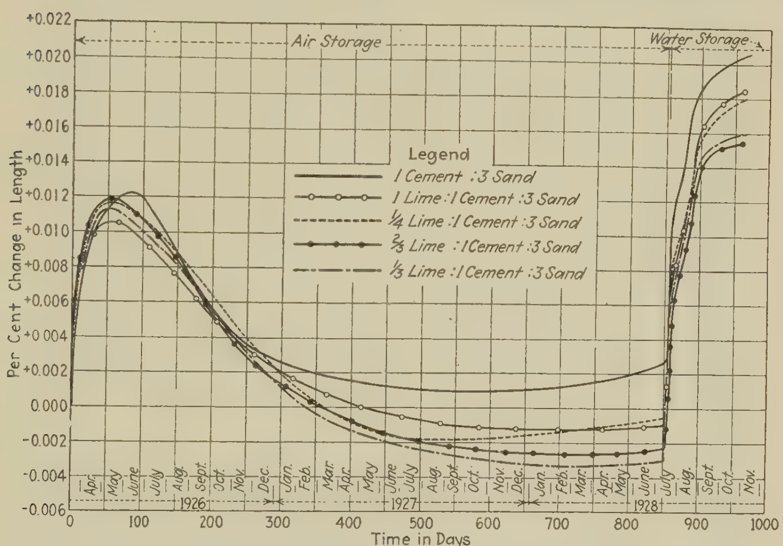

FIG. 8. CHANGES IN LENGTH OF BRICK PIERS—1926 SERIES.

absorbed by the brick prior to the initial set of the mortar, as the brick were not fully saturated when laid and were doubtless capable of absorbing considerable quantities of water in a short period of time. This left the mortar in a very dry condition. During the hardening process the cement drew upon the moisture stored in the brick, and a delayed chemical action took place resulting in the swelling of the colloids surrounding the cement grains. This, of course, was accompanied by an expansion of the mortar material causing an elongation of the piers. Finally, as the moisture in the piers evaporated and chemical actions were retarded, shrinkage began.

The elongation of the piers was greatest for the one laid with plain cement mortar and least for the pier using the mortar of high lime content, but the difference is not marked, showing that for the lime used, the lime

content does not materially affect the expansion. It is instructive to note that, if the elongation, varying from 0.011 to 0.012 per cent, takes place entirely within the mortar joints, it represents a mortar expansion of about 0.07 per cent, or roughly 1 in. per 100 ft.

Soon after reaching the maximum expansion in air, all piers contracted rather rapidly until about 10 months old. Thereafter they continued to contract, though at a much slower rate, but practically no shrinkage occurred after 24 months of air storage. The net lengths of the piers at this time were but little different from their original lengths, the pier with no lime having a net expansion of 0.001 per cent and the pier with one-third as much lime as cement having a net contraction of 0.003 per cent, values which are small, in comparison with the original expansions of the piers. When first kept under water spray following 28 months air storage all piers exhibited a rapid increase in length, but this rate of expansion slackened considerably after 2 months of water storage. As is indicated by the figure, the actual expansion of the piers accompanying watersoaking is practically the same for one as for another, but the expansion from the original length after 105 days of watersoaking is greatest for the pier with the plain cement mortar and least for the pier laid up with mortar containing two-thirds as much lime as cement. Their net expansions of 0.020 per cent and 0.015 per cent respectively, correspond to values of 1.5 and 1.1 in. per 100 ft. of mortar if all of the above expansion is considered to occur in the mortar joints.

The marked difference in the action of the bars and piers while stored in dry air as shown by a comparison of Figs. 6 and 8 makes it clear that the behavior of a mortar cast in a non-absorbent mold is no criterion to the behavior of the same mortar when employed in brick work.

Mortar Tests, 1927 Series.—These tests were made to determine the effect upon volume changes in mortars of (1) character of surrounding medium, and (2) type of mortar. Six mortars were tested. Five of these were made of one volume of cementing material to three volumes of sand, the cementing materials being portland cement, portland cement with 5 per cent clay, portland cement with 10 per cent clay, portland cement with 10 per cent high magnesian lime, and a patented plastic cement. The sixth mortar was a 1:5 mix (by volume), the portland cement containing 10 per cent clay. Where clay was used it was intimately ground with the cement prior to making the mortar. Sacramento River sand was used throughout, and measurements of clay and lime were by weight. All mortars were mixed to the same consistency as measured by the flow table.

Mortar-bar and brick-pier specimens were made from each of the above 6 mortars. The piers were of common Ramillard brick, a medium-grade, coarse-grained brick (15 per cent absorption). The joints were ½ in. thick. One-third of the bars of each mortar were cast in non-absorbent wooden molds which were removed after 48 hr.; one-third were cast in absorbent burned clay covered molds 1 in. thick lined with

dampened muslin, the molds being removed after 48 hr.; and the remaining bars were cast in similar absorbent burned clay molds and were left in the molds. All piers and bars have been stored continuously in air at 50 per cent relative humidity.

Fig. 9 shows the results for the three groups of bars made of the plastic cement mortar. It is seen that the bars with the absorbent molds left on exhibited an initial expansion of 0.004 per cent during the first 12 days (probably due to moisture being fed back to the bars by the absorbent molds during that period), and then began gradually to contract. After 375 days, when the rate of shrinkage had become very small, the contraction amounted to 0.096 per cent, or 1.1 in. per 100 ft. It will be noted that neither of the two groups of bars without molds exhibited a

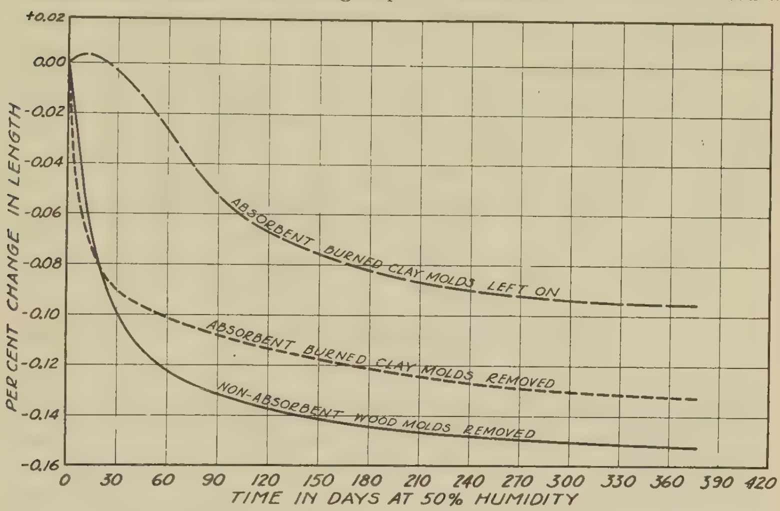

FIG. 9. SHRINKAGE OF MORTAR BARS, 1927 SERIES, PLASTIC CEMENT GROUP.

corresponding initial expansion, but both groups began immediately to shrink. The bars cast in the non-absorbent wood molds contracted the most, their shrinkage during the first year amounting to 0.153 per cent, more than two-thirds of which occurred during the first month. The bars cast in the absorbent molds and then removed therefrom exhibited contractions intermediate to the bars of the other two groups, as might be expected.

For each of the 1:3 mortars, the contractions of the three groups of specimens bear the same relation to each other as for the plastic cement groups.

Fig. 10 shows the changes in length of the bars of the 6 mortars cast in the non-absorbent wooden molds which were removed after 48 hr. Consulting the curves it appears that all bars contracted rapidly for the first 20 to 60 days and then continued to contract at a gradually lessening

rate. About 75 per cent of their shrinkage of the first year occurred during the initial 2-months period. None of the bars had attained volumetric equilibrium at the end of one year, but as indicated by the diagrams, the rate of increase of shrinkage is small. The 1:5 mix with 10 per cent clay showed a contraction of 0.092 per cent or 1.1 in. per 100 ft. at the end of 375 days. The 1:3 mix using plastic cement exhibited the largest

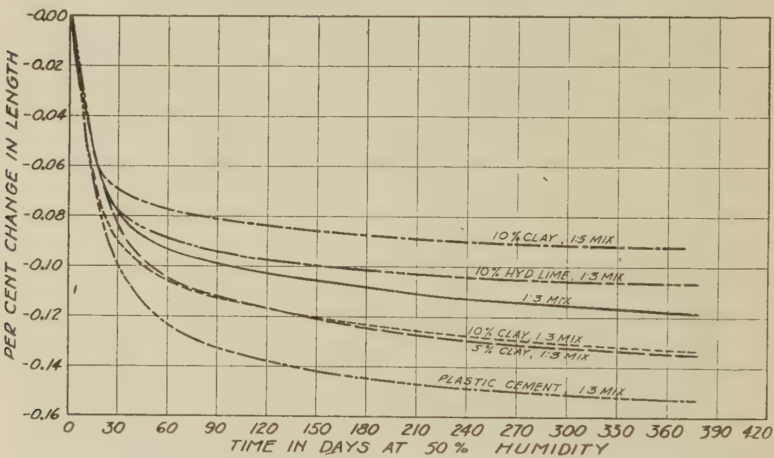

FIG. 10. SHRINKAGE OF CEMENT-LIME MORTAR BARS, 1927 SERIES, NON-ABSORBENT MOLDS REMOVED.

Table 1—Per Cent Contraction of Mortar Bars, 1927 Series, After 375 Days in Air

Mortar	Per Cent Contraction		
	Non-absorbent Wood Molds Removed	Absorbent Burned Clay Molds Removed	Absorbent Burned Clay Molds Left On
Plain 1:3 mix	0.119	0.102	0.072
5 per cent Clay in 1:3 mix	0.135	0.114	0.085
10 per cent Clay in 1:3 mix	0.133	0.122	0.095
10 per cent Hydrated Lime in 1:3 mix	0.106	0.094	0.080
Plastic Cement, 1:3 mix	0.153	0.134	0.096
10 per cent Clay in 1:5 mix	0.092	0.109	0.087

shrinkage, this amounting to 0.153 per cent, or 1.8 in. per 100 ft. during the 375-day period. The 1:3 mixes containing 5 per cent and 10 per cent of clay contracted more than the plain 1:3 mix, whereas the 1:3 mix with 10 per cent of hydrated lime contracted less than the plain 1:3 mix. Table 1 shows the contractions after 375 days in air for all three conditions as regards molds and for all six mortars.

From a study of Table 1 it will be seen that for the bars cast in the absorbent molds, those of the 1:3 mix with 10 per cent clay contracted

more than corresponding bars with 5 per cent clay. For all groups, the bars of 1:3 mix with 10 per cent clay contracted more than the corresponding bars of 1:5 mix, and also contracted more than the bars of 1:3 mix with 10 per cent hydrated lime.

The action of the brick piers of the 1927 Series is shown by the curves of Fig. 11, which give the changes in length as the average of changes in

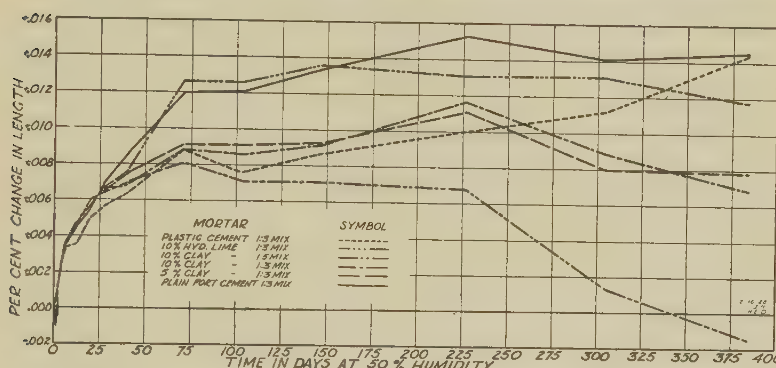

FIG. 11. CHANGES IN LENGTH OF BRICK PIERS, 1927 SERIES.

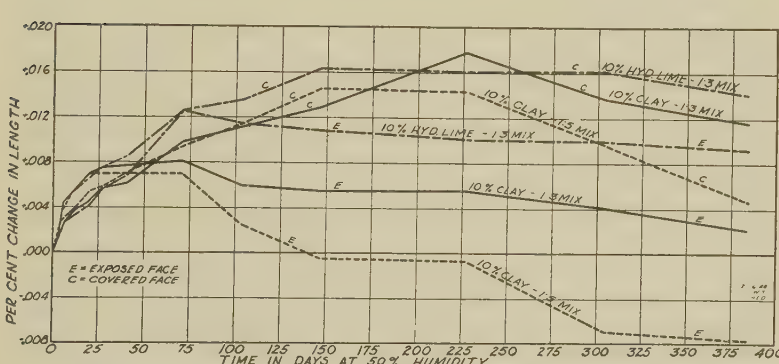

FIG. 12. COMPARISON OF EXPANSIONS OF EXPOSED AND UNEXPOSED FACES OF BRICK PIERS, 1927 SERIES.

exposed and opposite unexposed faces. These piers, like those of the 1926 Series, at first expanded at a rapid rate, but the expansion continued for a much longer period. The pier laid with 1:3 plastic cement mortar exhibited a continual elongation up to the last observation at 384 days, when the expansion was 0.014 per cent. The 1:5 mix with 10 per cent clay attained its maximum elongation of 0.008 per cent earlier than did the other piers, and this maximum was the smallest of the series. The

other piers reached their maximum expansion at about 225 days at values from 0.011 to 0.015 per cent. The latter expansion, which is the largest of those observed, corresponds to an expansion of 1.1 in. per 100 ft. of mortar joints if it be considered to occur entirely within the mortar joints. It will be observed that the unmodified portland cement mortar shows the maximum expansion. From the trend of the curves it appears as though the plastic cement may eventually exceed this maximum value, however. From Fig. 11 it will be seen that the pier laid in 1:3 mortar with 10 per cent clay expanded slightly more than the corresponding pier with mortar containing 5 per cent clay and considerably more than the pier laid in a 1:5 mortar with 10 per cent clay.

Fig. 12 shows the difference between the changes in length of the exposed face and the opposite covered face of three of the piers. For each pier shown the covered face expanded from 20 to 60 per cent more than the exposed face. This was no doubt due to the more rapid drying out of the exposed face. From a study of the figure it will be seen that the exposed faces attained their maximum expansions in about 70 days, while the covered faces continued to expand for periods two to three times as long.

Cement and Lime Mortars, 1928 Series.—These tests are being made on piers of Port Costa brick, laid in three types of mortar, a different mortar for each pier. Each mortar is composed of 2 parts of cementing material to 3 parts of Sacramento River sand, measured by volume. The cementing materials for the 3 mortars are: (*a*) equal parts of portland cement and high magnesian hydrated lime; (*b*) high magnesian hydrated lime without cement; and (*c*) equal parts of portland cement and high calcium lime putty. The piers are stored in air at 50 per cent relative humidity, three sides of each pier being covered with waterproof building paper and the fourth side being exposed to the air. Steel gage plugs ½ in. in dia. spaced 20 in. apart vertically, extend from the exposed face through the pier to the covered face opposite thereto. The period of observation for these piers extends over 170 days, the first observations being at the age of 1 day.

All of the piers began to expand as soon as constructed, as did those of the earlier series. The pier laid with the cement and lime putty mortar attained its maximum expansion of 0.013 per cent at the age of 2 months, the changes in length since that time being negligible. The pier using the hydrated lime mortar expanded quite rapidly for the first 20 days at which time its expansion amounted to 0.018 per cent. This pier continued to expand at a more gradual rate for the remainder of the observation period, the expansion amounting to 0.033 per cent at 170 days. The pier laid with the cement and hydrated lime mortar increased in volume considerably more than the other piers of this series, the expansion amounting to 0.021 per cent at 20 days and 0.050 per cent at 170 days. A review of the data indicates that under the present storage conditions in air a somewhat greater expansion may be expected for this latter pier but that very little additional expansion is likely to occur in the pier using

the straight hydrated lime mortar. It seems probable that the pier laid with the cement-lime putty mortar will soon begin to contract.

A comparison of the above expansions with those shown in Fig. 11 for the piers of the 1927 series indicates that for the same age, the expansions of the pier using the cement-hydrated lime mortar are about 5 times the expansions of the 1927 piers and that for the pier of the 1928 series laid with the hydrated lime mortar the expansion is about 3 times that for the 1927 piers. For the pier using the cement-lime putty mortar, the expansion is about the same as for the 1927 piers.

Effect of Water-Cement Ratio upon Volumetric Changes of Cement Mortars.—These tests are being made on mortars of two mixes to determine the effect of variations in the water-cement ratio upon the volumetric changes. The materials are Santa Cruz portland cement and Sacramento River sand. The proportions by volume are 1:2 and 1:6.2. For each of these mixes 3 water-cement ratios were used as shown in Table 2.

Table 2—Consistencies and Water-Cement Ratios of Mortar Mixes

	1:2 Mix			1:6.2 Mix		
Water-Cement ratio	0.67	1.40	1.86	1.54	3.78	4.12
Slump in inches	0	2	4	0	2.5	9

The molds for the bars of rich mortars were removed after 1 day and the molds for the bars of lean mortar were removed after 3 days, it being necessary in the latter case to delay removal of the molds because of low early strength of the bars. Observations were begun when the molds were removed. After removal of the molds one-half of the bars of each mortar were stored continuously in dry air at 50 per cent relative humidity, and the remainder were stored continuously in water.

The diagrams of Fig. 13 indicate the changes in length of the several groups during the interval of 4 months. Considering the mortar bars of rich mix which were continuously wet, it will be observed that those with the lower water-cement ratio expanded about 50 per cent more than did those with the higher water-cement ratios, the respective expansions being 0.038 per cent and 0.024 per cent.

Considering the bars of rich mix stored in dry air, it is seen that the drier mix with a water-cement ratio of 0.67 shrunk 0.106 per cent and the wet mix with a water-cement ratio of 1.86 shrunk 0.136 per cent.

From an examination of the diagrams for bars of the 1:6.2 mix, it appears that the water-cement ratio of the lean mix has no appreciable influence either upon the magnitude of the expansion accompanying immersion or upon the shrinkage accompanying air storage.

Effect of Waterproofing Admixtures upon Volumetric Changes of Cement Mortars.—To determine the effect of waterproofing admixtures upon the volumetric changes of cement mortars, tests were made upon seven groups

of bars. For five of these the basic mix was one part Santa Cruz portland cement to three parts of Niles top gravel, a coarse sand having a fineness modulus of 3.2, the proportions being by weight. One group of these specimens was made without any admixture. For a second group of specimens of this mix, stearic acid to the amount of 2½ per cent by weight of the cement was added. To the three other groups, patented waterproofing liquids were added in the proportions recommended by

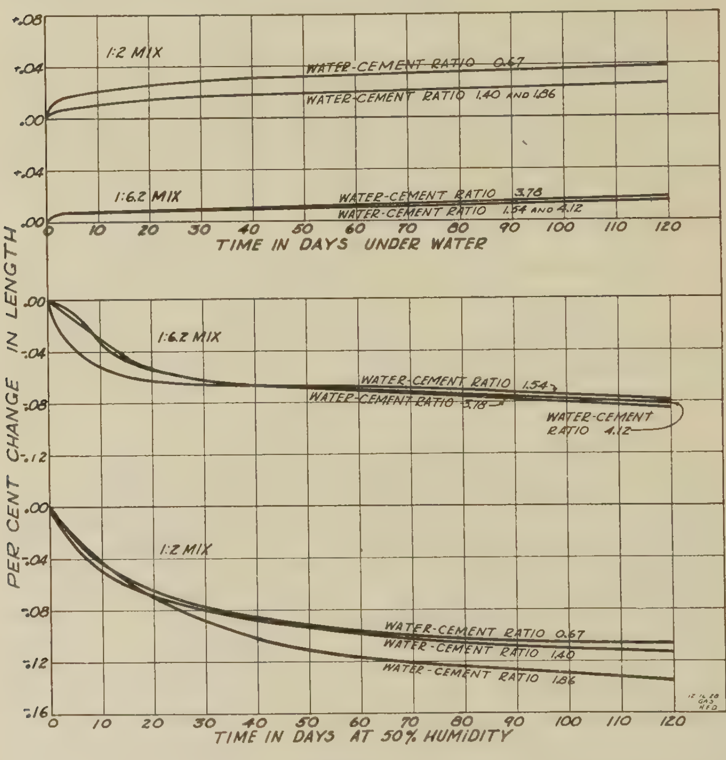

FIG. 13. EFFECT OF WATER-CEMENT RATIO UPON VOLUME CHANGES IN MORTARS.

the manufacturers. The two additional groups, making the seven, were prepared by using special waterproofing cements in the ratio of one part cement to three parts of Niles top gravel. For all mortars, the consistency was such as to produce a slump of about 5 in.

The molds were stripped at the age of three days. The bars were then subjected to the following storage conditions: Water for 7 days, air at 75 per cent relative humidity for 21 days, air at 45 per cent relative humidity for 14 days, water for 7 days, drying oven at 110 deg. C. for 7

days, water for 8 weeks, air at 75 per cent relative humidity for 15 months. Table 3 gives the test results, net contractions referring to contractions from the original lengths.

A study of Table 3 shows that the mortars in which waterproofing admixtures are included exhibit greater volumetric changes than does the unmodified portland cement mortar. It will be seen that in the oven dry condition, waterproofing cement No. 2 shows more than double, and waterproofing liquid No. 3 nearly double, the shrinkage exhibited by the normal portland cement mortar.

After being oven dried and later immersed in water for a considerable period, all mortar bars, including those of normal portland cement, tend to reach volumetric equilibrium while still considerably shorter than their original length. In other words, a permanent shrinkage seems to have occurred which is greater in all cases for the waterproofed mortars than for the normal cement mortar.

TABLE 3—EFFECT OF WATERPROOFING ADMIXTURES UPON VOLUMETRIC CHANGES

Waterproofing Material	Maximum Per Cent Contraction in Dry Air for 35 Days	Net Per Cent Contraction After 7 Days in Water	Maximum Per Cent Contraction Oven Dry	Net Per Cent Contraction After Eight Weeks in Water	Net Per Cent Contraction After 15 Months in Air
Normal Santa Cruz Cement	0.058	0.021	0.087	0.019	0.077
Waterproofing Cement No. 1	0.077	0.030	0.104	0.037	0.114
Waterproofing Cement No. 2	0.128	0.069	0.186	0.088	0.130
Waterproofing Liquid No. 1	0.093	0.031	0.106	0.026	0.094
Waterproofing Liquid No. 2	0.068	0.022	0.099	0.026	0.080
Waterproofing Liquid No. 3	0.081	0.038	0.155	0.040	0.083
Stearic Acid	0.095	0.051	0.119	0.053	0.102

Crushed Granite Concrete.—These tests are made upon bars from a mix of 1 part Colton portland cement to 5 parts by weight of graded aggregate composed entirely of crushed granite, nearly all passing the 1½-in. sieve and 9 per cent passing the 100-mesh sieve. The ratio of volume of water to that of cement was 1.0, and the average slump was 3.2 inches. Specimens are kept continually at 70 deg. F., and those stored in air are subjected to a constant relative humidity of 50 per cent.

The behavior of a group of specimens of this series after immersion in water at the age of 2 days is shown by Fig. 14, the lower curve giving the per cent of increase in length, and the upper curve giving the per cent of increase in weight. It is seen that the increase in length continues at a gradual and ever decreasing rate for nearly two years after which time the change is practically negligible. The maximum increase in length in this period amounts to nearly 0.020 per cent which corresponds to nearly ¼ in. per 100 ft. About one-half of this total change occurred within the first three months. The increase in weight of these bars is seen to take place much more abruptly than the increase in length, nearly three-quarters of the total increase in weight of 2.1 per cent after two years,

occurring within the first three weeks after immersion. After 2 years the weight appears still to be slowly increasing.

The diagrams of Fig. 15 showing the corresponding changes in a group of specimens cured in damp sand for 28 days and thereafter stored in air have the same general trend as those of Fig. 14, but the air shrinkage is over three times the corresponding expansion in water and the decrease in weight in air is over twice the corresponding increase in weight in

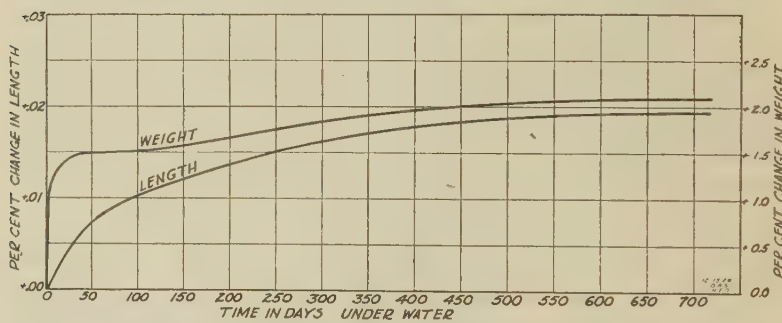

FIG. 14. CHANGES IN LENGTH AND WEIGHT OF GRANITE CONCRETE BARS UNDER WATER.

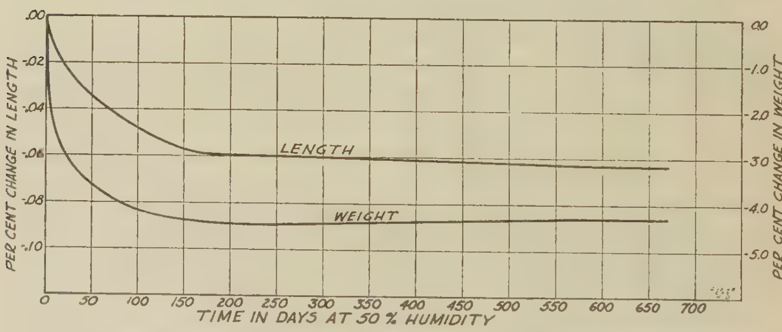

FIG. 15. CHANGES IN LENGTH AND WEIGHT OF GRANITE CONCRETE BARS, AIR STORAGE.

water. There is little decrease in length after the first seven months of air storage. At the end of two years, the shrinkage amounts to about ¾ in. per 100 ft. It is of interest to note that this is less than one-half of the corresponding shrinkage of the 1:3 mortar for which shrinkage diagrams were shown in Fig. 6.

In order to determine the effect of alternate wetting and drying upon expansion and contraction, five groups of specimens have been under observation, the times of alternate air and water storage being varied among the several groups. Specimens of these groups were cured

in damp sand for 28 days and were then immersed in water for 7 to 10 days prior to the beginning of observation.

Fig. 16 shows the changes in length produced by alternate air-dry periods of 6 weeks and water-soaking periods of 1 week. From a study

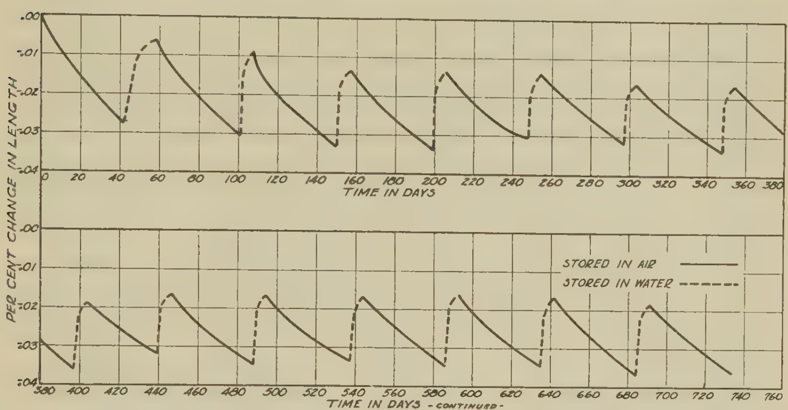

FIG. 16. CHANGES IN LENGTH OF GRANITE CONCRETE BARS CAUSED BY 6-WEEKS AIR DRYING.

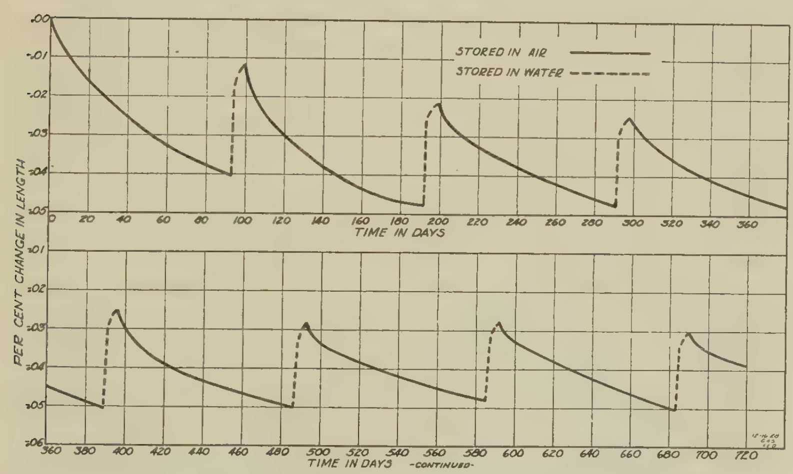

FIG. 17. CHANGES IN LENGTH OF GRANITE CONCRETE BARS CAUSED BY 3-MONTHS AIR DRYING.

of the curve it appears that the maximum net contraction is becoming greater with time but that after the first three drying periods, the actual shrinkage during each 6-weeks drying period has changed very little. During the first three cycles, all the contractions and the expansions are

greater than those produced subsequently, indicating that up to an age of about 5 months the changes in length are greater than those which may be expected later.

Fig. 17 is a similar diagram for the group of bars subjected to 3-months air-drying periods followed by 1-week water-soaking periods. The maximum contraction referred to the original length has remained constant after the fourth drying period (at about one year), but the actual shrinkage during each drying interval is gradually becoming less. A minimum change will perhaps soon occur and begin repeating itself, similar to the minimum change now repeating itself in the 6-weeks drying interval. It will be observed from a study of Figs. 16 and 17 that after a prolonged period in air there is a residual shrinkage when the concrete is again water soaked and the longer the drying period, the greater this residual shrinkage becomes. This is further substantiated by the diagrams of Fig. 7 for the 1:3 mortar showing what appears to be a practically constant residual shrinkage of about one-third the maximum, after water soaking for two months, the prior drying period having been seven months.

Other groups of specimens have been subjected to alternate drying periods of 5 days or 10 days and water-soaking periods of 2 days, while still another group has been subjected to alternate drying periods of 3 weeks and water-soaking periods of 1 week. After 4 repetitions for these specimens there is a rest period, when the specimens are stored in damp sand for a period of 2 months for the 5-day and 10-day air-storage groups and 3 months for the 3-weeks air-storage group, followed by a 7-day period of water-soaking, when the process of alternate drying and soaking is repeated four times more.

From a review of the data for these latter groups it appears that for all series of four alternations other than the first series, the changes of length (expansion and contraction) for the first alternation are less than for the second, the second less than the third, and the third less than or equal to the fourth. While there appears to be this increase in the changes for any one series of four alternations, yet there is a slight decrease in the magnitude of these changes for the later series.

Effect of Fines on Shrinkage of Crushed Granite Concrete.—In order to determine the effect of the finer particles of the aggregate upon shrinkage and expansion, two series of tests were made upon groups of regular 3 x 3 x 40-in. specimens for which the aggregate was composed entirely of a graded crushed granite, practically all of which passed a ¾-in. sieve. For both of these series the ratio of the volume of water to that of cement was 1.0. For the first series the cement ratio was kept constant, the mix being 1 part Colton cement to 4 parts of mixed aggregate and the slump being 4 to 6 in. for the several groups, depending upon the percentage of fine material. In the second series the consistency of the mix was kept constant, the slump being 3 in. and the mix varying from 1:4.5 to 1:5.2 for the several groups, depending upon the per cent of fine material. As received, the fine aggregate (passing ¼-in. sieve) for the first series con-

tained by weight 25 per cent of particles passing the 100-mesh sieve and 16 per cent of particles passing the 200-mesh sieve; and for the second series contained 26 per cent passing the 100-mesh sieve and 9 per cent passing the 200-mesh sieve. For all mixes the volumetric ratio of the fine to the coarse aggregate was 2 to 3.

The specimens of one group of each series contained the fine aggregate as received, that is, the fine aggregate contained all the material passing the 100-mesh sieve; the specimens of the second group of each series contained aggregate passing the 100-mesh sieve to the extent of 6 per cent of the weight of the fine aggregate; and the specimens of the

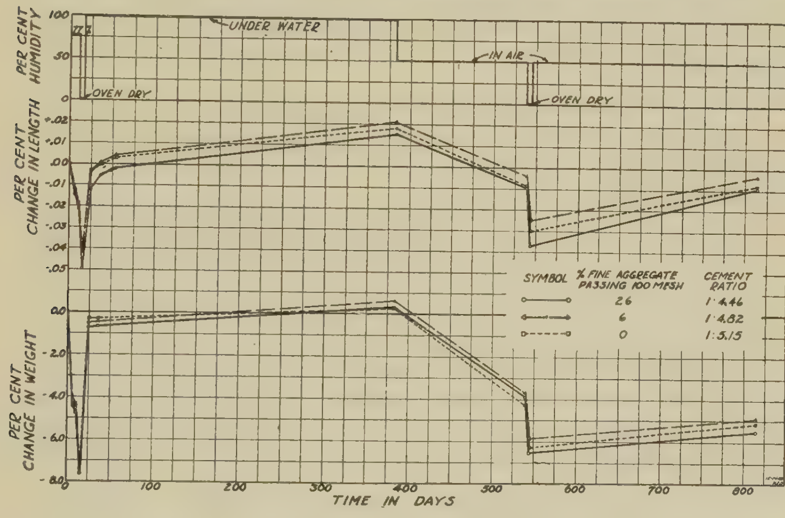

FIG. 18. EFFECT OF FINE AGGREGATE UPON CHANGES IN LENGTH OF GRANITE CONCRETE BARS.

third group were made by using aggregate from which all the fine material passing a 100-mesh sieve had been removed. For all groups of both series the specimens were cured for four days in damp sacks prior to the removal of molds, at the end of which period observations of changes in length and weight were begun. They were then stored in air for about 2 weeks, oven-dried at 105 deg. C. to constant weight, immersed in water for approximately 1 year, stored in air for about 5 months, dried a second time to constant weight, and then stored in air for about 9 months.

Fig. 18 shows the results of the tests of the second series, the upper diagrams giving in per cent the changes in length for each of the three groups and the lower diagrams giving in per cent the changes in weight. The first thing that catches the eye is the striking similarity between the upper and lower groups of curves, which demonstrates how closely change of weight varies with change in volume. Secondly, the percentage of

fines apparently has no marked influence either upon the magnitude of the shrinkage when in the dry state or upon the magnitude of the expansion after the prolonged period of immersion. Lastly, the shrinkage at the end of the second oven-drying period is somewhat less than at the end of the first oven-drying period, though prior to the second period of oven-drying the specimens were air-dried for 5 months.

It is instructive also to note that the maximum shrinkage, amounting to about 0.05 per cent, is about the same as that exhibited by the group stored for two years continuously in air after curing 28 days in damp sand, illustrated by the diagram of Fig. 15, and that the maximum swelling at the end of the period of immersion, amounting to about 0.02 per cent is not much different from that exhibited by the group stored continuously in water for a period of 2 yr. as shown by the diagram of Fig. 14.

Comparing the data for the 2 series, it is found that the variations in weight are nearly the same but that the specimens of the first series, for which the cement ratio is the larger, exhibit the greater shrinkage and the smaller swelling.

Volumetric Changes in Gravel Concrete.—Tests to determine the volumetric changes which occur in gravel concrete accompanying variations in moisture have been in progress for four years, the concrete being composed of Niles gravel and Santa Cruz portland cement. This gravel, coming from the Coast Range, consists chiefly of granite, quartzite, diorite, and sandstone pebbles, and finer sand particles composed of a mixture of decomposed argillaceous and igneous materials including about 30 per cent quartz. The gravel was screened into four sizes and then recombined in predetermined proportions to give four different gradations of aggregate, a parallel series of tests being made for each of these gradations. The specimens are 3 by 3 by 40-in. bars.

For Series 1 the aggregate is composed entirely of small pebbles nearly all between the sizes ⅜ in. and ⅛ in. Its fineness modulus is 4.83, and its surface modulus is 4.74.

For Series 2 the aggregate is composed of 1 part of coarse sand between the sizes ⅛ in. and $\frac{1}{16}$ in. and 3 parts of large pebbles between the sizes 1 in. and ⅜ in. The fineness modulus is 6.16 and the surface modulus is 3.73.

For Series 3 the aggregate is composed of 38 per cent large pebbles (1 in. to ⅜ in.), 25 per cent small pebbles (⅜ in. to ⅛ in.), 20 per cent coarse sand (⅛ in. to $\frac{1}{16}$ in.), and 17 per cent fine sand (below $\frac{1}{16}$ in. with 1 per cent passing the 100-mesh sieve). Fineness modulus 5.04; surface modulus 7.05.

For Series 4 the aggregate is composed of 19 per cent large pebbles (1 in. to ⅜ in.), 62.5 per cent small pebbles (⅜ in. to ⅛ in.), 10 per cent coarse sand (⅛ in. to $\frac{1}{16}$ in.), and 8.5 per cent fine sand (below $\frac{1}{16}$ in.). Fineness modulus 4.93; surface modulus 5.90.

Within each series there are 4 groups of specimens varying from each other as regards richness of mix. The mixes are as follows: Group A,

1:2; Group B, 1:3; Group C, 1:4.5; and Group D, 1:6. Also a group of neat cement specimens was made.

The consistency of the mix was such as to produce a slump of ½ to 1 in. Specimens were cured under water for 28 days prior to the beginning of observations. For the neat cement specimens and for groups of the first three series the testing program was as follows, beginning with observations in the saturated condition at the end of approximately 28 days: oven-drying to constant weight; storage under observation in air at constant temperature of 78 deg. F. and constant relative humidity of 45 per cent for period of 6 weeks; storage in atmosphere subjected to the normal variations of temperature and humidity without observation; storage in air at 78 deg. F. and 75 per cent relative humidity for 17 weeks;

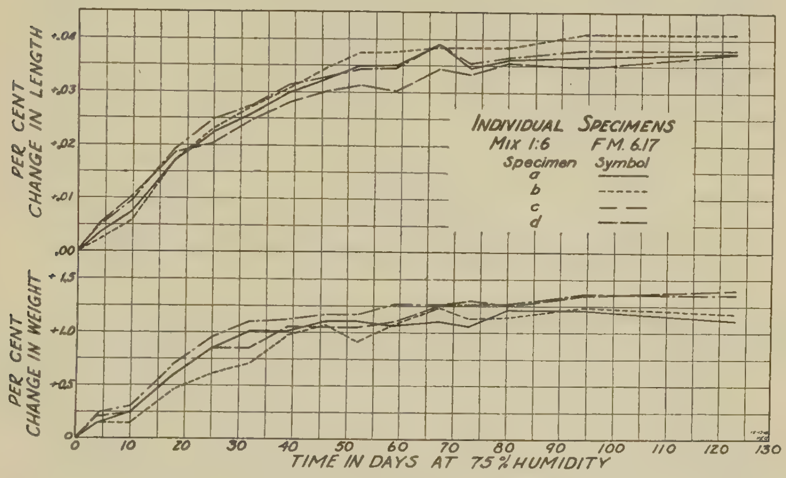

FIG. 19. EXPANSION AND ABSORPTION OF GRAVEL CONCRETE BARS, MEDIUM COARSE, 1:6 MIX.

oven-drying to constant weight; storage under observation in air at 78 deg. F. and 95 per cent relative humidity for 17 weeks; water-soaking for 3 weeks; at rest without observation in normal atmosphere for 44 weeks; oven-drying to constant weight; water-soaking for 3 weeks; storing in air at 70 deg. F. and 50 per cent relative humidity for 31 weeks; storing in water spray at 70 deg. F. for 28 weeks.

The specimens of Series 4 were cast after the tests at 45 per cent relative humidity had been run, and the schedule of tests for this series is the same as for the first three series beginning with the tests at 75 per cent relative humidity. Fig. 19 shows the changes in length (upper diagrams) and changes in weight (lower diagrams) for each of the 4 bars of Series 2, Group D (coarse sand and large pebbles, 1:6 mix) during the process of moisture absorption and expansion while in air at 75 per cent relative humidity after having been oven-dried.

It will be noted that, in general, there is close agreement between the several bars, and further that there is close correlation between change in length and change in weight. These relations are typical of those found to exist for other groups of the several series.

Fig. 20 shows graphically the behavior of Series 1, Group A (small

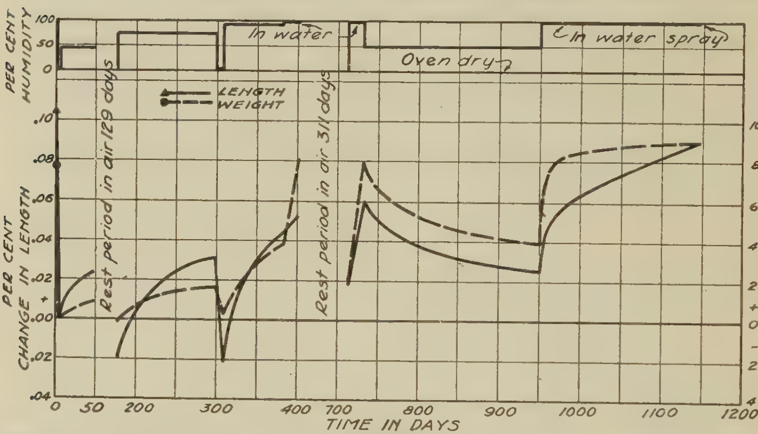

FIG. 20. CHANGE IN LENGTH AND WEIGHT OF GRAVEL CONCRETE BARS, 1:2 MIX, SERIES 1 (SMALL PEBBLES).

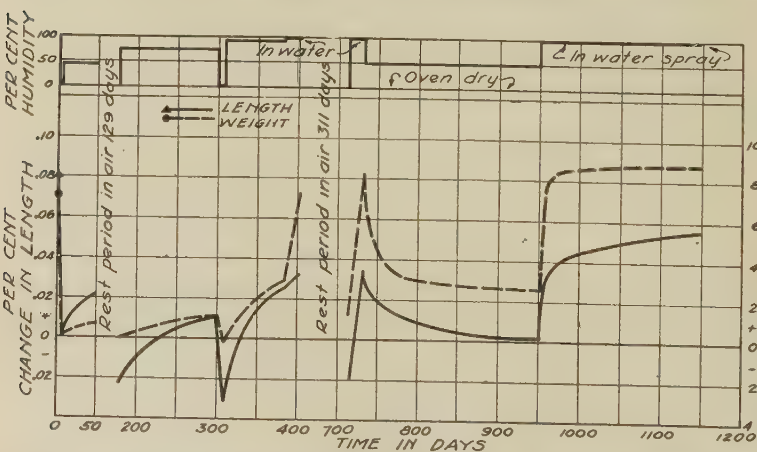

FIG. 21. CHANGE IN LENGTH AND WEIGHT OF GRAVEL CONCRETE BARS, 1:6 MIX, SERIES 1 (SMALL PEBBLES).

pebbles, 1:2 mix) since the beginning of the tests. The diagram near the top of the figure shows the humidity conditions during storage except that zero humidity indicates an oven-drying period and 100 per cent humidity indicates a water-soaking period or a water-spraying period, as noted. The full-line curves of the lower portion of the figure show the

relation between change of length and time, the changes being reckoned from the first oven-dry length; and the broken line curves of the lower portion of the figure indicate in a similar manner the changes in weight. The gaps in these curves indicate the rest periods.

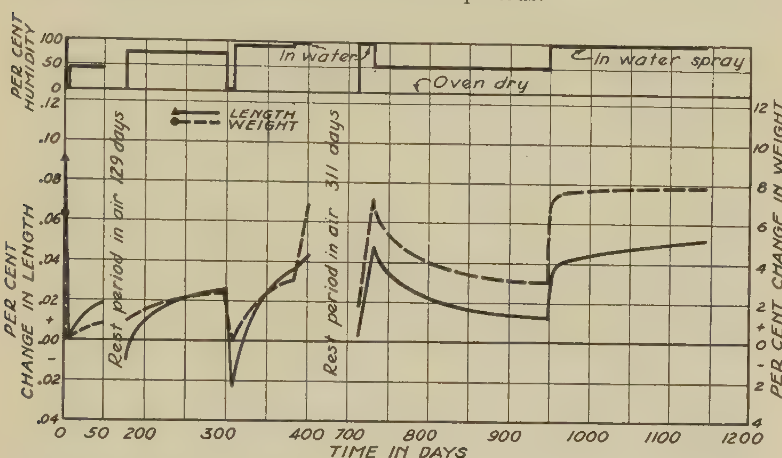

FIG. 22. CHANGE IN LENGTH AND WEIGHT OF GRAVEL CONCRETE BARS 1:4.5 MIX, SERIES 2 (LARGE PEBBLES AND COARSE SAND).

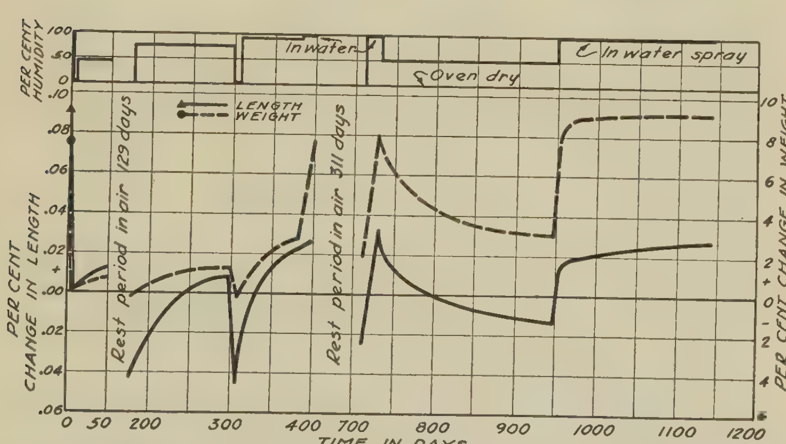

FIG. 23. CHANGE IN LENGTH AND WEIGHT OF GRAVEL CONCRETE BARS. 1:3 MIX, SERIES 3 (GRADED AGGREGATE WITH SLIGHT EXCESS OF FINES).

A study of the figure shows that, when stored in a humid atmosphere, these specimens increase in weight and in length, the rate being rapid at first and then gradually decreasing until it becomes very small or practically zero. It is evident that the higher the relative humidity, the more

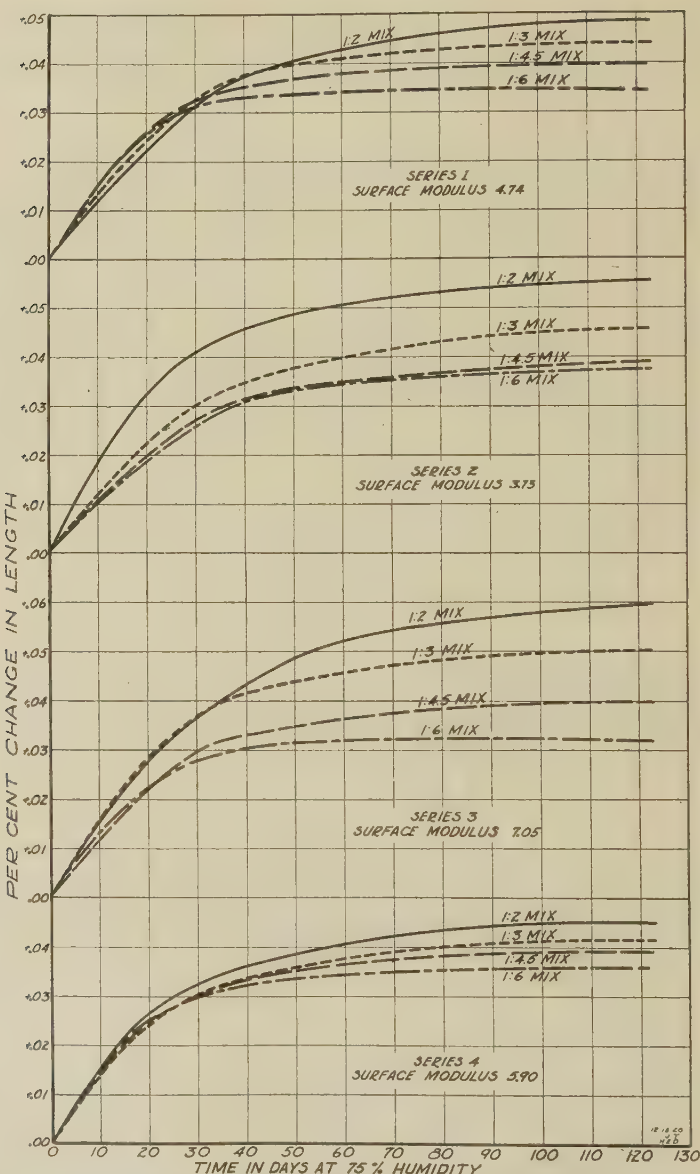

FIG. 24. INFLUENCE OF CEMENT RATIO UPON CHANGE IN LENGTH OF GRAVEL CONCRETE BARS, OVEN-DRY TO 75 PER CENT RELATIVE HUMIDITY.

rapid is the initial rate of absorption and expansion and the greater is the magnitude of the absorption and expansion before a state of equilibrium is approached. It is also clear that the changes in moisture content are not alone responsible for changes in length, as illustrated by the fact that shrinkage from a water-soaked state to an oven-dry state at the beginning of the tests (left end of diagram) is augmented by further shrinkage when oven-dried following both the 45-per-cent and the 75-per-cent humidity periods, while the initial oven-dry weight corresponds very closely with the oven-dry weights following the periods of 45- and 75-per-cent relative humidities. Furthermore, when at the end of the 95-per-cent relative humidity period the specimens are water-soaked, the increase in weight is very considerable but the increase in length is slight.

It is also instructive to observe that the initial shrinkage from wet to oven-dry is materially greater than the following expansion from oven-dry through the 95-per-cent period to the water-soaked condition; and that, in turn, this latter expansion is considerably larger than the expansion occurring between the oven-dry and water-soaked stages succeeding the 10-months rest period in air. This points to the contraction from a wet to a dry state or the expansion from a dry to a wet state becoming less with age. It is significant that the water-soaked weights just prior to and just following the rest period of 10 months in air agree very closely with the original weight, while the corresponding lengths are about mean between the original water-soaked length and the original oven-dry length.

Figures 21, 22, and 23 respectively give corresponding diagrams for Group D, Series 1 (1:6 mix, small pebbles); Group C, Series 2 (1:4.5 mix, large pebbles and coarse sand); and Group B, Series 3 (1:3 mix, graded aggregate with slight excess of fines). A comparison with the previous figure shows a marked similarity between the 4 sets of curves.

Fig. 24 shows the effect upon gravel concrete bars of storage in air at 75 per cent relative humidity subsequent to oven-drying. The diagrams of the figure indicate that, regardless of the gradation of aggregates, the expansion increases directly with the cement ratio. Fig. 25 shows the same data rearranged with diagrams of like richness of mix grouped together to show the influence of gradation of aggregate upon the changes in length. From a study of the figure it appears that, within the limits of the tests, the gradation of the aggregate does not have a large influence upon the changes in length, although in general it appears that the expansion decreases as the surface modulus increases.

Figs. 26 and 27 show diagrams corresponding to those of Figs. 24 and 25 for specimens stored in air at 45 per cent relative humidity subsequent to oven-drying. The figures indicate that for this condition the expansion increases as the cement ratio increases and decreases as the surface modulus of the aggregate increases.

Corresponding diagrams for specimens stored in a water spray subsequent to air drying at 50 per cent relative humidity are shown in

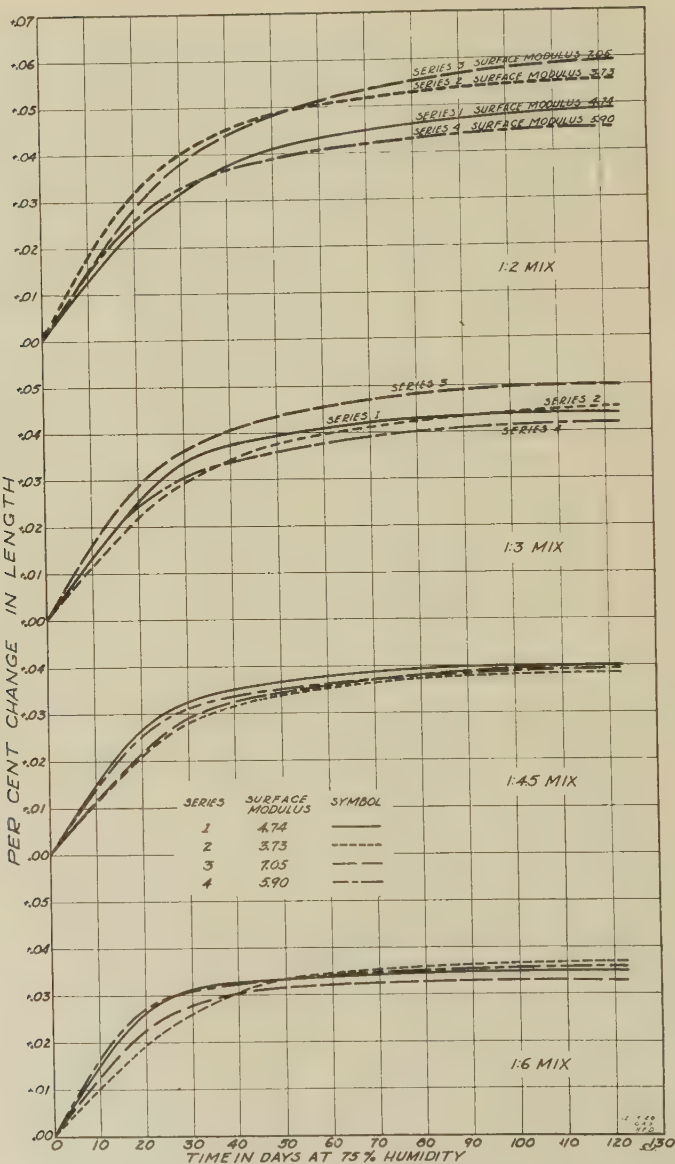

FIG. 25. INFLUENCE OF GRADATION OF AGGREGATE UPON CHANGE IN LENGTH OF GRAVEL CONCRETE BARS, OVEN-DRY TO 75 PER CENT RELATIVE HUMIDITY.

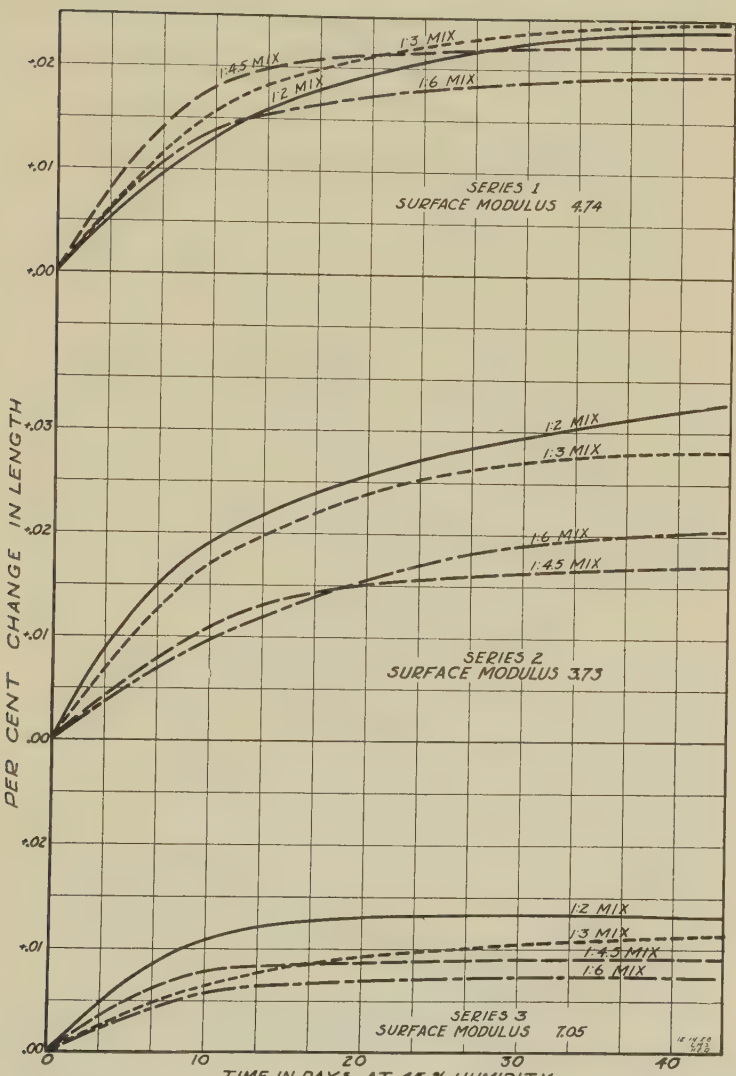

FIG. 26. INFLUENCE OF CEMENT RATIO UPON CHANGE IN LENGTH OF GRAVEL CONCRETE BARS, OVEN-DRY TO 45 PER CENT RELATIVE HUMIDITY.

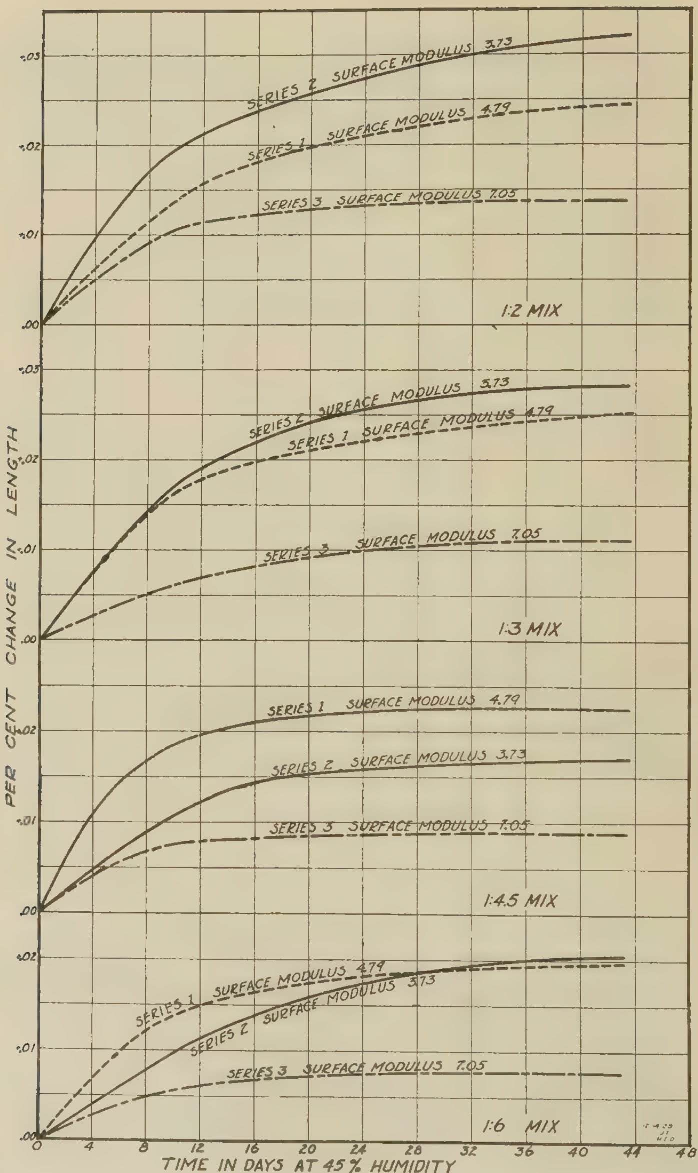

FIG. 27. INFLUENCE OF GRADATION OF AGGREGATE UPON CHANGE IN LENGTH OF GRAVEL CONCRETE BARS, OVEN-DRY TO 45 PER CENT RELATIVE HUMIDITY.

Figs. 28 and 29. Consulting Fig. 28 it will be observed that, in general, the expansion increases with the cement ratio; while from Fig. 29 it will be noted that, in general, the expansion increases as the surface modulus

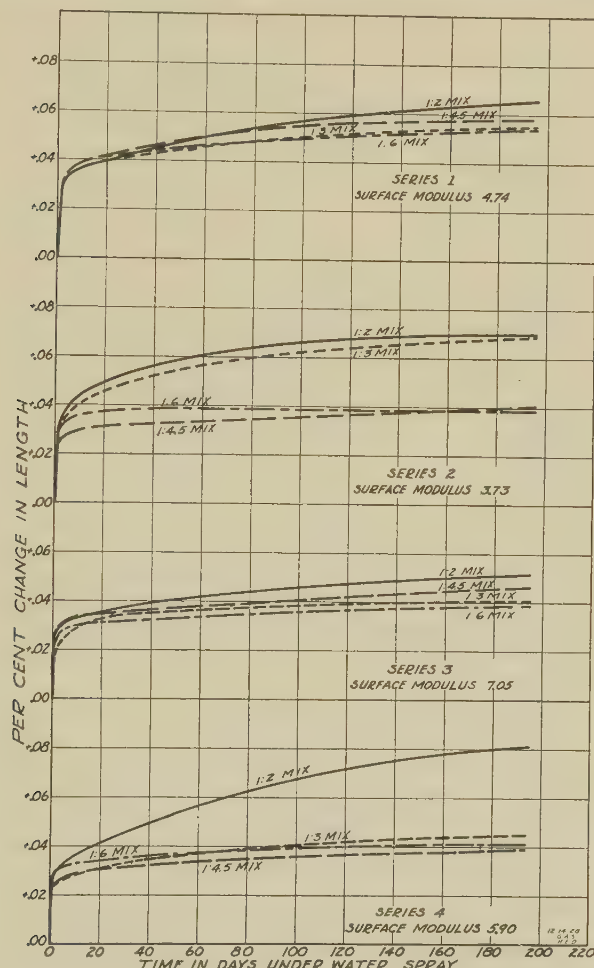

FIG. 28. INFLUENCE OF CEMENT RATIO UPON CHANGE IN LENGTH OF GRAVEL CONCRETE BARS, AIR-DRY TO WATER-SPRAY.

decreases, that is, the expansions are greater for the coarse aggregates (this is further demonstrated by Fig. 33).

To render a simple comparison of the expansions and contractions of the several groups, these values have been plotted against surface

modulus of the aggregates of the several series. Fig. 30 shows the contraction that took place at the beginning of the tests when the specimens were oven-dried from a water-soaked condition at the approximate age

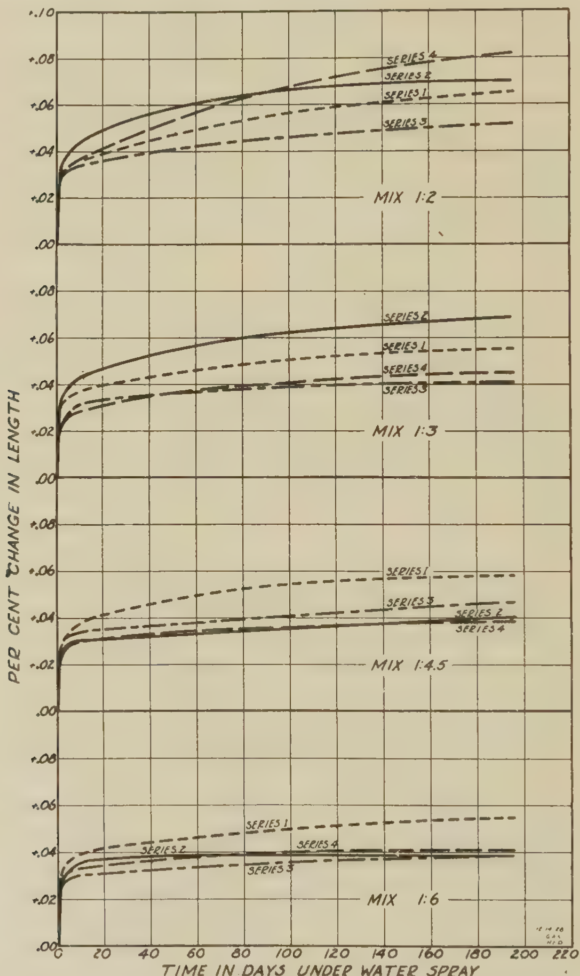

FIG. 29. INFLUENCE OF GRADATION OF AGGREGATE UPON CHANGE IN LENGTH OF GRAVEL CONCRETE BARS, AIR-DRY TO WATER-SPRAY.

of 28 days. For the leaner mixes and, in general, for the richer mixes, it appears that the contraction varies inversely with the surface modulus, or in other words, those specimens which contain fine sand contracted less

during this initial drying than did those containing no fine material. It may also be observed from this figure that the contraction varies directly with the cement ratio.

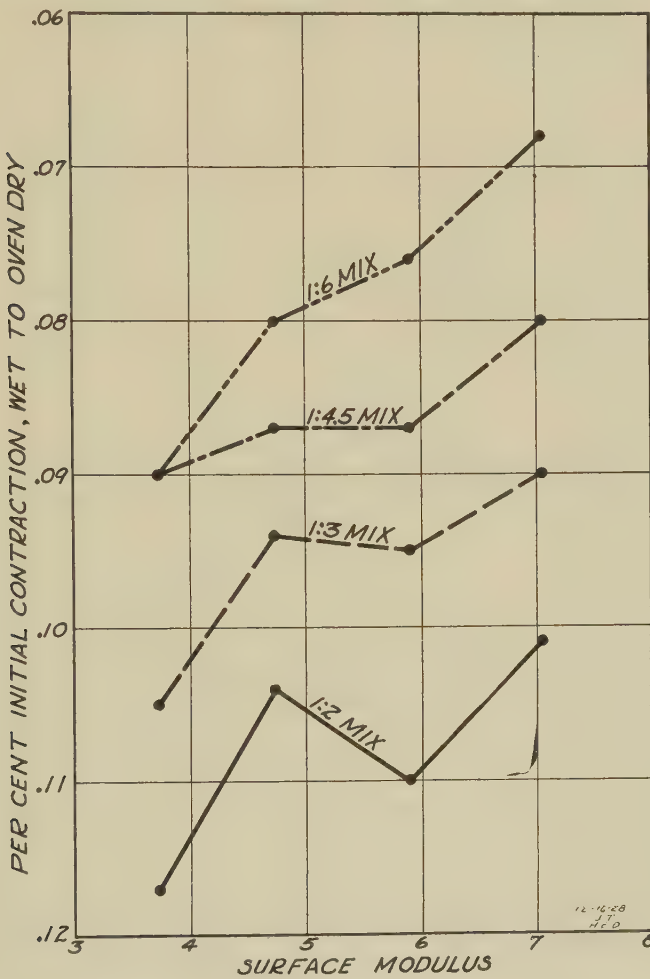

FIG. 30. INITIAL CONTRACTION OF GRAVEL CONCRETE BARS, WET TO OVEN-DRY.

Fig. 31 shows similar diagrams for the expansion which took place when the specimens were stored in air at 45 per cent humidity. Again it appears that the gradation of the aggregate influences the change in

length, the expansion being less for specimens of high surface modulus than for those of low surface modulus.

The expansions are greater for the rich mixes than for the lean ones,

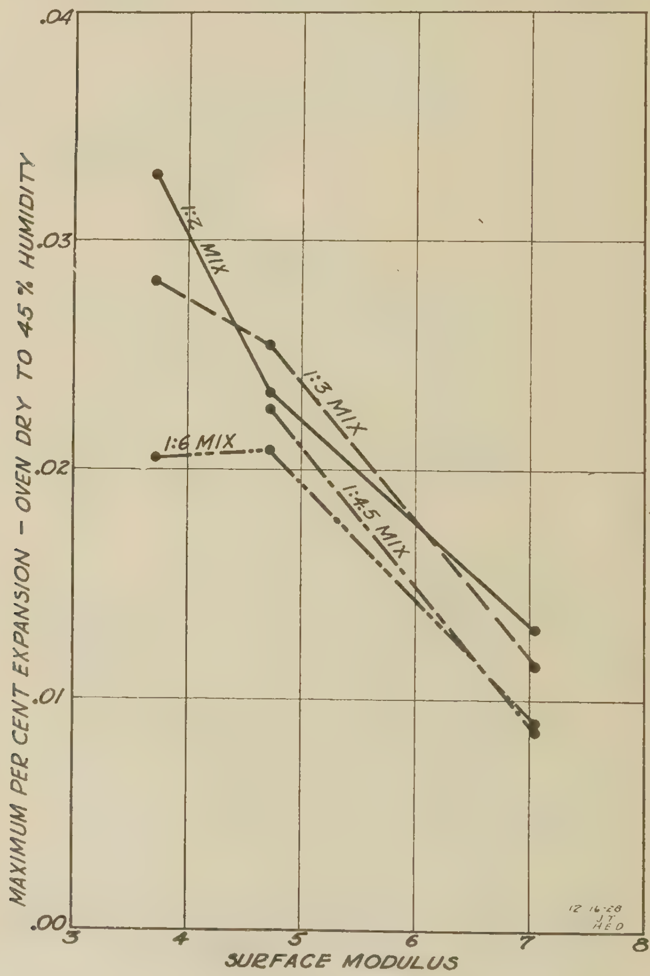

FIG. 31. EXPANSION OF OVEN-DRIED GRAVEL CONCRETE BARS IN AIR AT 45 PER CENT RELATIVE HUMIDITY.

but it will be observed that for this condition of storage, the surface modulus exerts a greater influence upon the expansion than does the richness of mix.

Fig. 32 illustrates in a similar manner the expansions that took place during the period when the specimens were stored in air at 75 per cent relative humidity after a rest period of 4 months in normal air followed

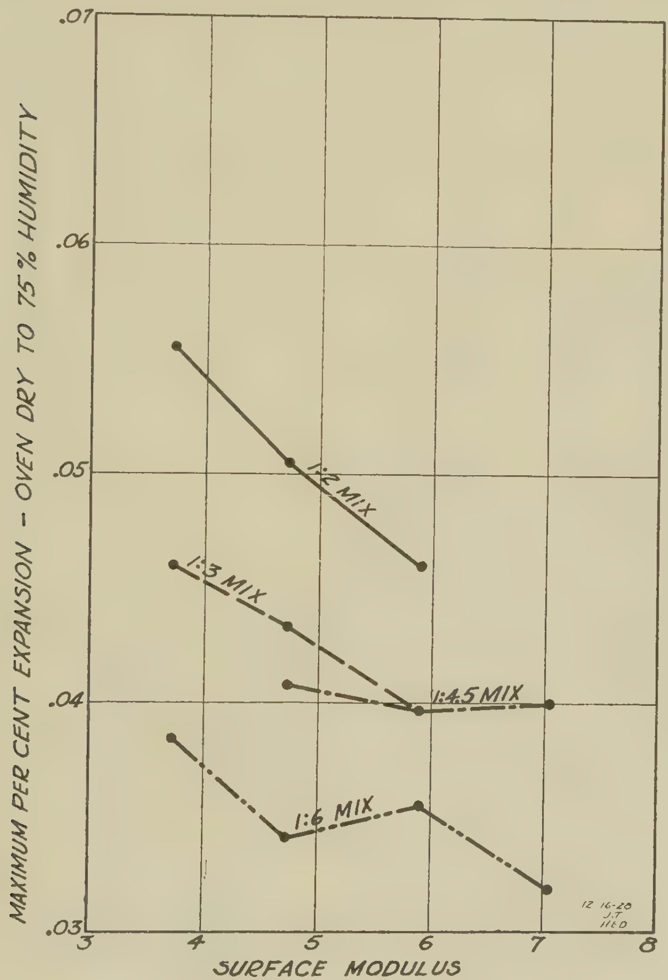

FIG. 32. EXPANSION OF OVEN-DRIED GRAVEL CONCRETE BARS IN AIR AT 75 PER CENT HUMIDITY.

by a second oven-drying. From the data presented, it appears that the expansion increases with the cement ratio and that it decreases as the surface modulus increases.

Fig. 33 shows the changes in length that occurred during the period when the specimens were stored in a water spray subsequent to air dry-

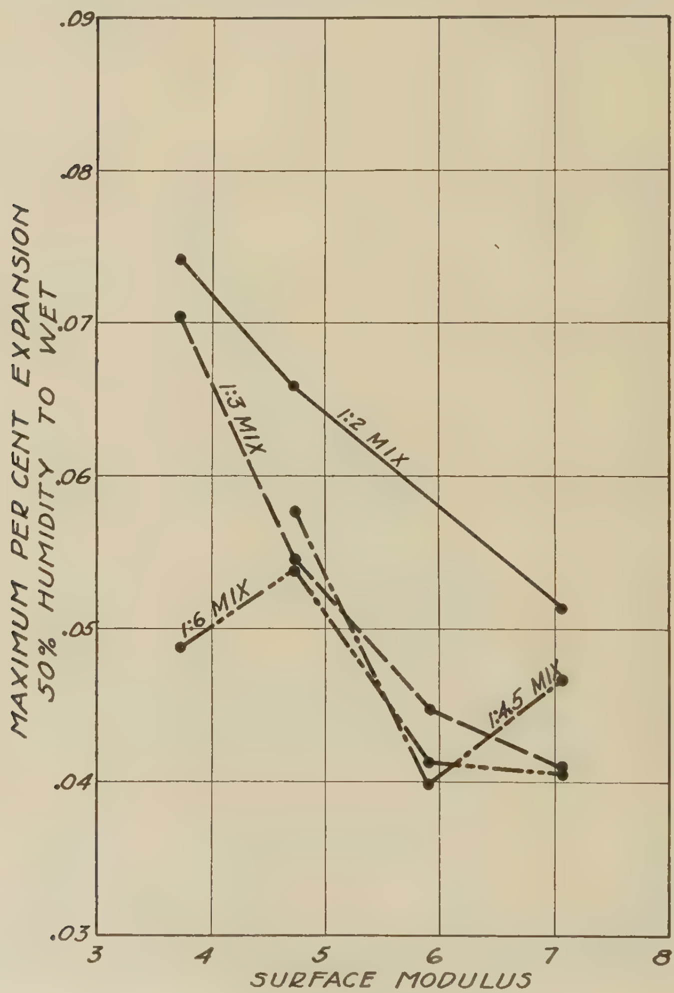

FIG. 33. EXPANSION OF AIR-DRIED GRAVEL CONCRETE BARS IN WATER-SPRAY.

ing at 50 per cent humidity. Here again it appears that the expansion decreases as the surface modulus increases.

SUMMARY OF RESULTS

Cement-Lime Mortars, 1926 Series.—These tests are being made upon mortars for which the basic mix by volume is 1 part portland cement to 3 parts of fine river sand. To this basic mix has been added high calcium hydrated lime in varying proportions up to a volume equal to that of the cement. The specimens are mortar bars and brick piers. The investigation here reported covers 32 months. During this period some of the bars have been stored continuously in air while the remainder have been 7 months in air and 25 months in water. The piers have been 28 months in air and 4 months in water spray. Following are some of the results:

(1*a*) Mortar bars in air decreased in length rapidly at first, the shrinkage amounting to 0.14 to 0.16 per cent (about 2 in. per 100 ft.) at one year. Though the indications are that shrinkage is still in progress, the rate is so small after one year as to make the additional shrinkage quite insignificant. (See Fig. 6.)

(1*b*) There is no evidence that the lime content affected the shrinkage to an appreciable degree, though the shrinkage is smallest for the cement mortar without lime. (See Fig. 6.)

(2*a*) Mortar bars when stored in air for 7 months and thereafter stored in water do not return to their original length but show a net shrinkage of ⅓ to ½ the shrinkage prior to immersion. About 70 per cent of the recovery of length occurs within the first 20 hours. The bars reach a state of volumetric equilibrium in about three months after immersion. (See Fig. 7.)

(2*b*) When volumetric equilibrium is reached, the expansion accompanying immersion is least and the net shrinkage is, in general, greatest for the plain cement mortar, and the expansion accompanying immersion is greatest and the net shrinkage is least for the mortar containing the largest lime content; but the difference is not large. (See Fig. 7.)

(3*a*) Brick piers in air expand during the first two or three months. The expansion varies from 0.013 to 0.015 in. per 10-ft. height of pier. If it be assumed that this takes place entirely within the mortar joints, it represents a mortar expansion of about 1 in. per 100 ft. (See Fig. 8.)

(3*b*) The plain cement mortar pier shows the largest expansion in air, and the pier laid in mortar with the greatest lime content shows the least; but the difference is not marked. (See Fig. 8.)

(3*c*) After reaching their maximum expansion in air, the brick piers shrink continuously and quite uniformly until they regain their original length when 12 to 14 months old. Further changes are quite gradual. (See Fig. 8.)

(3*d*) When volumetric equilibrium is reached, the plain cement mortar pier shows a net expansion of 0.001 per cent, while the other piers show net contractions of 0.001 to 0.003 per cent, values which are small as compared to the maximum expansion in air which occurred within the early months of air storage. (See Fig. 8.)

(4*a*) Brick piers previously stored in air for 28 months and then kept wet continuously show a marked increase in length for the first two months of water soaking. Volumetric equilibrium has not been reached after 4 months in water. (See Fig. 8.)

(4*b*) The expansion resulting from water soaking these piers after 28 months of air storage is practically the same for all piers, irrespective of lime content. In the 4 months of water soaking it amounts to about 0.018 per cent. (See Fig. 8.)

(4*c*) The expansion from the original length, or net expansion, does not appear to bear any consistent relation to the lime content, but it is worthy of note that the maximum expansion from the original length which amounts to 0.020 per cent, is for the pier with plain cement mortar. (See Fig. 8.) If it be assumed that this takes place entirely within the mortar joints, it represents a mortar expansion of 1.5 in. per 100 ft.

(5) The volumetric changes of a mortar bar cast in a non-absorbent mold and stored in air is no criterion to the volumetric changes that may be expected to occur in the same kind of mortar when used as the joint material in brick work. (See Figs. 6 and 8.)

Cement Mortars, 1927 Series.—These tests are being made upon 6 mortars. Five of these are composed of 1 part cementing material to 3 parts of Sacramento River sand, measured by volume, the cementing materials being Santa Cruz cement, cement with 5 per cent clay, cement with 10 per cent clay, cement with 10 per cent high magnesian hydrated lime, and a plastic cement. The sixth mortar is a 1:5 mix, the cement containing 10 per cent clay. The cement and clay were intimately ground together before incorporating with the sand. The specimens are mortar bars and brick piers. One-third of the bars of each mortar were cast in non-absorbent wooden molds which were removed after 48 hr.; one-third were cast in absorbent burned clay molds which were removed after 48 hr.; and the remaining bars were cast in absorbent burned clay molds which were left in place. All piers and bars have been stored continuously in air at 50 per cent relative humidity. Three sides of each pier are inclosed in waterproof building paper and the fourth side is exposed to the air. The results are as follows:

(1) All mortar bars with molds removed shrunk quite rapidly for the first month. The shrinkage was more gradual from thence onward. (See Figs. 9 and 10.)

(2) Mortar bars left in absorbent molds expanded slightly for the first two weeks. Thereafter, they shrunk but at a more gradual rate than did the bars with molds removed. (See Fig. 9.)

(3) After one year of air storage, the rate of shrinkage is quite slow. From a comparison with the results of other tests it appears probable that further changes will be small. (See Figs. 9 and 10.)

(4) The shrinkage of the bars with non-absorbent molds removed is about 50 per cent greater than the shrinkage of the bars with absorbent molds left on. The bars with absorbent molds removed are intermediate in volumetric changes to the other two groups. (See Fig. 9 and Table 1.)

The only exception to this is the 1:5 mix using 10 per cent clay, for which the shrinkage of the bars with non-absorbent molds removed is intermediate to the shrinkage of the bars with absorbent molds left on and absorbent molds removed.

(5) The mortar bars made by using the plastic cement shrunk more than any of the others. (See Fig. 10 and Table 1.)

(6) For all types of molds, the 1:3 mortars with 10 per cent clay contracted more than did the 1:5 mortars with 10 per cent clay. (See Fig. 10 and Table 1.)

(7) For all types of molds, the 1:3 mortars with 10 per cent clay contracted more than the 1:3 mortars with 10 per cent hydrated lime. (See Fig. 10 and Table 1.)

(8) For the bars cast in absorbent molds, those made of the 1:3 mix with 10 per cent clay contracted more than those made of the 1:3 mix with 5 per cent clay (see Table 1). For the bars with non-absorbent molds removed, there was practically no difference between these two mortars. (See Fig. 10 and Table 1.)

(9) The brick piers expanded for about 7 months, after which all except the one laid with plastic cement mortar began gradually to shrink. The latter pier was still expanding after 13 months. (See Fig. 11.)

(10) The unmodified cement mortar expanded the most (see 1926 Series also) and the 1:5 mix with 10 per cent clay expanded the least, the actual expansions being 0.0152 per cent and 0.0080 per cent respectively. If all of the expansion is assumed to occur in the mortar joints, these values correspond respectively to 1.1 in. and 0.6 in. per 100 ft. of mortar joints (see Fig. 11). It is possible that in the process of time the expansion of the plastic cement mortar pier may exceed the expansion of the plain cement mortar pier.

(11) After 12 months in air, the pier laid with the 1:5 mix with 10 per cent clay reached its original length and thereafter showed some contraction, whereas all the other piers exhibited a net expansion, even though for some of them this expansion was decreasing somewhat. (See Fig. 11.)

(12) The pier laid with the 1:3 mix with 10 per cent clay expanded a little more than that made of the 1:3 mix with 5 per cent clay. (See Fig. 11.)

(13) The pier laid with the 1:3 mix with 10 per cent clay expanded about 40 per cent more than the pier laid with the 1:5 mix with 10 per cent clay. (See Fig. 11.)

(14) For all piers, the exposed face expanded much less than the opposite covered face (see Fig. 12). The exposed face reached its maximum expansion in about 70 days, whereas the opposite covered face continued to expand for a much longer period. (See Fig. 12.)

Mortars—1928 Series.—These tests are being made on common brick piers laid with 3 varieties of mortar. Each mortar is composed of 2 parts of cementing material to 3 parts of Sacramento River sand, measured by volume. The cementing materials for the 3 mortars are (*a*) equal parts

of portland cement and high magnesian hydrated lime; (*b*) high magnesian hydrated lime without cement; and (*c*) equal parts of portland cement and high calcium lime putty. The piers are stored in air at 50 per cent relative humidity, three sides of each pier being covered with waterproof building paper and the fourth side being exposed to the air. The period of observation extends over 170 days. All of the piers began to expand as soon as constructed, as did those of the earlier series. The results follow:

(1) The pier using the cement-lime putty mortar reached its maximum expansion of 0.013 per cent after 2 months. Changes since that time have been negligible.

(2) The pier using the hydrated lime mortar was still expanding at the end of 170 days, but the rate of expansion was very small. At 170 days its maximum expansion amounted to 0.033 per cent.

(3) The pier laid up with the cement-hydrated lime mortar was still expanding at a fairly rapid rate after 170 days, at which time its expansion amounted to 0.050 per cent.

Effect of Water-Cement Ratio on Volumetric Changes.—These tests are being made on bars of cement mortar of 2 mixes and 3 consistencies. Santa Cruz cement and Sacramento River sand were used, the proportions being 1:2 and 1:6.2 by volume. For each of the mixes, mortars of dry, medium, and wet consistencies exhibited the respective slumps of 0, 4½, and 9 in. Half the bars of each group were stored continuously in water and the remainder were stored continuously in air at 50 per cent relative humidity. Following are some of the results:

(1) Of the 1:2 mortars kept continuously wet, that having the smallest water-cement ratio has the largest expansion (see Fig. 13).

(2) Of the 1:2 mortars stored in dry air, the larger the water-cement ratio, the larger the resulting contraction (see Fig. 13).

(3) For the 1:6.2 mortars, there is no apparent effect of consistency of mix upon either the expansion when stored in water or upon the shrinkage when stored in dry air (see Fig. 13).

(4) At a given age the expansions of the 1:2 mortars stored in water are considerably greater than those of the 1:6.2 mortars; and likewise the contractions of the 1:2 mortars stored in dry air are considerably greater than those of the 1:6.2 mortars (see Fig. 13).

Effect of Waterproofing Admixtures Upon Volumetric Changes.—These tests are being made on cement mortars using a coarse sand (fineness modulus 3.2), the proportions being 1:3 by weight. In some cases, waterproofing cements were used while for other mortars, waterproofing compounds were added to a normal portland cement mortar. One group of specimens is made of the portland cement mortar without admixtures. The consistency of all the mortars was such as to produce a slump of about 5 in. The specimens are mortar bars which have been subjected to various storage conditions including oven drying and water soaking and air storage at 45 and 75 per cent relative humidities. The results follow:

(1) All of the waterproofed mortars tested exhibit greater volumetric changes than does the normal portland cement mortar under similar conditions (see Table 3).

(2) In the oven-dry condition, some of the waterproofed mortars show about double the shrinkage exhibited by the normal portland cement mortar (see Table 3).

(3) After oven drying and subsequent water soaking for a period of two months, all the mortar bars show a net shrinkage which is least for the normal cement mortar and greatest for one of the waterproofing cements (see Table 3).

(4) After a further storage of 15 months in air, the normal portland cement mortar still exhibits the smallest shrinkage.

Crushed Granite Concrete.—These tests are being made on bars composed of 1 part cement to 5 parts of crushed granite graded below the 1½ in. size, 9 per cent of the aggregate passing a sieve having 100 meshes to the inch and measurements being by weight. The water-cement ratio was 1 by volume. The tests have extended over a period of about 2 years. Some of the specimens have been stored continuously in water; the remainder were cured in damp sand for 28 days, since which time some have been stored continuously in air and others have been alternately stored in air and in water, the time between alternations varying for the several groups. The results follow:

(1) The specimens stored in water with the passage of time increase both in length and in weight. The increase in length takes place at nearly a uniform rate for the first 2 or 3 months and at the end of 2 years the state of volumetric equilibrium seems nearly to have been reached. The increase in weight is more abrupt, most of it occurring within the first 15 days, but the indications are that thereafter there is a slow increase accompanying the increase in length (see Fig. 14).

(1*a*) The change in length within the two-year period amounts to 0.020 per cent. The corresponding change in weight is 2.1 per cent (see Fig. 14).

(2) Specimens stored in air after having been water-soaked both shrink and lose weight, the rate of shrinkage and dehydration becoming less with the passage of time (see Fig. 15).

(2*a*) At the end of 2 years, the shrinkage is about 0.06 per cent or roughly ¾ in. per 100 ft. and the loss in weight is over 4 per cent.

(3) Within the first 6 months the loss in weight of the concrete in air at 50 per cent humidity is about 3 times as much as the gain of weight of concrete under water; similarly, the shrinkage in air is about 5 times the expansion in water for the same period (see Figs. 14 and 15).

(4) Under conditions of alternate air and water storage there is no evidence that the expansion accompanying immersion is materially different from the contraction accompanying the preceding period of air storage when the period of air storage is 10 days or less, but for periods of air storage of greater length it appears that the total expansion is less than the total contraction, leaving a residual shrinkage (see Figs. 14–17).

(4a) The maximum shrinkages from the initial water-soaked state are as follows (see Figs. 14–17):

Length of air storage periods	5 days	10 days	3 weeks	6 weeks	3 months
Length of water storage period	2 days	2 days	1 week	2 w.–1 w.	1 week
Shrinkage, per cent	0.007	0.020	0.025	0.037	0.050

(5) For bars subjected to 4 cycles of alternations of storage conditions and then allowed to rest in damp sand, the changes in length (expansion and contraction) for the first cycle are less than for the second, the second less than for the third, and the third less than or equal to the fourth.

(6) Bars subjected to alternating conditions of air and water storage undergo greater contractions and expansions at early ages than at later ages. In time the changes accompanying cycles of stress tend to become constant (see Figs. 16 and 17).

Effect of Fines Upon the Shrinkage of Crushed Granite Concrete.— These tests are being made upon concretes for which the aggregate (including all fine material) is crushed granite. They have extended over a period of 2 years. For all specimens the ratio of volume of water to volume of cement was 1. The ratio of fine (passing No. 4 sieve) to coarse aggregate (maximum size ¾ in.) was 2:3. For one series the fine aggregate contained 26 per cent of material which passed the No. 100 sieve and 9 per cent which passed the No. 200 sieve; for a second series the fine aggregate as received contained 25 per cent of material passing the No. 100 sieve and 16 per cent passing the No. 200 sieve. In one group of tests of each series the fine aggregate was used as received; in a second group all fines passing the No. 100 sieve were removed; and in a third group fines to the extent of 6 per cent by weight of the fine aggregate were incorporated in the mix. For the first series the mix was 1 cement to 4 mixed aggregate; for the second series the consistency was constant (slump 3 in.), the mix varying from 1:4.5 to 1:5.2.

Tests were begun on wet specimens when 4 days old, and test conditions were as follows: two weeks in air, oven-drying to constant weight, 8 months in water, 5 months in air, oven-drying to constant weight. The results follow:

(1) The percentage of fines (granitic material passing 100-mesh sieve) seems to have no appreciable effect upon the shrinkage when the concrete is dried nor upon the expansion when the concrete is water-soaked for a long period (see Fig. 18).

(2) Changes in length are accompanied by closely corresponding changes in weight (see Fig. 18).

(3) Specimens having the higher cement ratios in general show the larger shrinkages when oven-dried and the smaller net expansions when water-soaked for a prolonged period (see Fig. 18).

(3a) The extreme change in length for the full test period is about 0.07 per cent for the specimens of each series, but the contraction from the original length is about 0.05 per cent for the first series and about 0.04 per cent for the second series (see Fig. 18).

(3b) The extreme change in length is about ½ that observed for a gravel concrete of the same richness of mix but with materially less fines.

(4) At the termination of the second period of oven-drying the specimens were both heavier and longer than they were after the first oven drying, though the difference is not large.

Volumetric Changes of Gravel Concrete as Affected by Variations in Cement Ratio, Gradation of Aggregate, and Humidity.—These tests are being made on bars composed of gravel concrete, the gravel being screened into four sizes and recombined so as to give four different gradations. The tests are divided into 4 series differing as regards gradation of aggregate, and each series is divided into 4 groups differing from one another as regards richness of mix. The recombined aggregates for the 4 series vary from one of nearly ideal gradation but somewhat above Fuller's curve (surface modulus 7.1) to one composed almost entirely of pebbles and very coarse sand (surface modulus 3.7). The mix for the 4 groups in each series varies from 1:2 to 1:6 by volume. Neat cement bars have also been under observation.

The tests have extended over a period of nearly 4 years and the specimens have been repeatedly oven-dried and water-soaked, and have been stored in air at constant temperature and regulated humidity for varying periods of time. Tests were begun upon specimens cured in water for about 28 days. Following are some of the results so far obtained:

(1) When specimens of a given gradation and mix are oven dried and then are stored in an atmosphere of constant humidity and constant temperature, they expand and absorb moisture rapidly at first and then at a decreasing rate until finally a state of volumetric and weight equilibrium is reached. It appears that expansion and absorption cease at practically the same time. In general, the agreement between the several specimens of the same groups is very close (see Fig. 19).

(2) When concrete in the oven-dry state is stored in air, its rate of increase in length and in weight varies with the degree of humidity, and the magnitude of its expansion and absorption when equilibrium is reached is a function of the degree of humidity (see Figs. 20, 21, 22, and 23).

(3) Concrete cured in water for 28 days and then oven dried in the customary manner exhibits a shrinkage less than that which will ultimately be obtained if the concrete is stored in air for a considerable period and then is again oven dried, but the subsequent oven-dry weights agree closely with the original weight (see Figs. 20, 21, 22, and 23).

4. Furthermore, when concrete has passed through the stages mentioned above and after water soaking is stored in air for a long period of time and then is oven-dried, its weight is greater than the original oven-dry weight but its length may be greater or less than the original oven-dry length, depending upon the richness of the mix and the gradation of the aggregates. For aggregates of low surface modulus, the length is likely to be greater than the original oven-dry length and for aggregates of high surface modulus it is likely to be less than the original oven-dry

length; for rich mixes greater, and for lean mixes less (see Figs. 20, 21, 22, and 23).

(5) When concrete originally is cured for a normal length of time under water and is then subjected to successive stages of oven-drying and long periods of air-storage, it takes on a permanent shrinkage which is not recovered when it is again water-soaked by immersion though it returns practically to its original water-soaked weight. It therefore appears that over a long period of time, changes in weight do not run quite parallel with changes in length. (See Figs. 20, 21, 22, and 23.)

(6) The original shrinkage from wet to oven-dry is greater than the subsequent shrinkage from wet to oven-dry with intervening conditions as mentioned above (see Figs. 20, 21, 22, and 23).

(7) The general behavior of neat cement under like conditions of storage is similar to that of a rich concrete with aggregate of high surface modulus, though, of course, the magnitude of the changes is much greater.

(8) When oven-dry concretes containing aggregates of a given gradation but with varying richness of mix are stored in air at a constant humidity, the richer the mix, the greater the expansion and the greater the absorption (see Fig. 24).

(9) When concretes are cured for 28 days in water and then oven-dried in the customary manner, as previously explained, the shrinkage accompanying the drying is likely to be less for lean mixes than for rich ones and also less for aggregates of high surface modulus than for those of low surface modulus (see Fig. 30).

(9*a*) The limits of this initial shrinkage are as follows for the several mixes:

Mix..................	1:2	1:3	1:4.5	1:6
Per cent shrinkage.....	0.101–0.117	0.090–0.105	0.080–0.092	0.068–0.087

The greater value for each mix is for aggregate of low surface modulus and the lesser one is for aggregate of high surface modulus (see Fig. 30).

(10) When concretes are cured for 28 days in water, then oven-dried and stored in air at a normal humidity, the expansion which takes place is in general less for lean mixes than for rich ones and less for aggregates of high surface modulus than for those of low surface modulus (see Fig. 31).

(10*a*) At 45 per cent relative humidity the limits of these expansions are as follows for the several mixes:

Mix..............	1:2	1:3	1:4.5	1:6
Per cent expansion	0.0130–0.0330	0.0116–0.0283	0.0090–0.0226	0.0085–0.0200

(11) When oven-dry concretes of considerable age having a given richness but with aggregates of varying gradations are stored in air at a constant humidity, while they do not expand and absorb moisture at the same rate, the lean mixes behave with sufficient uniformity to indicate that the gradation of the aggregate has no large influence upon changes in length, whereas for the rich mixes, the expansion decreases as the surface modulus increases (see Fig. 32).

(11*a*) At 75 per cent relative humidity the limits of expansion for the several mixes are as follows (see Fig. 32):

Mix	1:2	1:3	1:4.5	1:6
Per cent expansion	0.046–0.055	0.040–0.046	0.040–0.041	0.032–0.038

(12) When concretes which have been in dry air for some time are placed in water spray, the resulting expansion is affected by both the richness of mix and by the surface modulus. For all mixes the expansion decreases as the surface modulus increases. Also the expansion is the greatest for the rich mixes. (See Fig. 33.)

(12*a*) The limits of expansion for the several mixes placed in water spray for 8 months after drying in air at 50 per cent relative humidity for 8 months are as follows (see Fig. 33):

Mix	1:2	1:3	1:4.5	1:6
Per cent expansion	0.051–0.074	0.041–0.070	0.040–0.058	0.040–0.054

(13) The maximum change in length that has developed since the beginning of the tests is greater for rich mixes than for lean ones. In a general way it appears also that the change in length is greater for aggregates of low surface modulus than for those of high surface modulus.

(14*a*) Following are the maximum observed changes in length for each group of each series:

Series	Fineness Modulus	Surface Modulus	Shrinkage, per cent			
			Group A, 1:2 Mix	Group B, 1:3 Mix	Group C, 1:4.5 Mix	Group D, 1:6 Mix
1	4.83	4.74	0.125	0.117	0.110	0.113
2	6 17	3.73	0.136	0.115	0.113	0.107
3	5.04	7.05	0.157	0.137	0.128	0.101
4	4.93	5.90	0.144	0.105	0.093	0.083

(14*b*) The maximum shrinkage of the neat cement is 0.404 per cent.

(15) The character of the aggregate has an important influence upon volumetric changes. The crushed granite concrete with all fines included has a surface modulus of about 20, with 26 per cent of fine aggregate passing a No. 100 sieve; Series 3 of the gravel concrete tests has a surface modulus of 7, with 1 per cent of fine aggregate passing the No. 100 sieve. In spite of this wide discrepancy in fines the gravel concrete has developed about double the shrinkage exhibited by the crushed granite concrete.

Prof. Hatt. W. K. HATT—The Laboratory for Testing Materials of Purdue University has been carrying on investigations similar to those by the authors of this paper since 1924. The same factors have been under investigation with the exception of the variation in quantity of water and the richness of mix and the effect of admixtures. We have added to the factors entering into the tests at the University of California the following:

(1) Study of volume changes of normal and special cements.
(2) Study of the effect of small amounts of reinforcing—from 0.20 to 0.35 per cent.
(3) Study of the ability of concrete to undergo deformations without surface fissures and the determination of the fatigue limit of concrete under various conditions of exposure.

TABLE I—EXPANSION AND CONTRACTION OF GRAVEL CONCRETE
1:2:3 Mix

(*a*) Contraction in Air

Gradation	Total Contraction at End of 400 Days in Air of Laboratory after 14 Days Initial Wet Burlap Curing, per cent	Fineness Modulus of Mix	W/C
Coarse	0.0480	5.69	0.80
Fine	0.0643	5.15	0.99

(*b*) Expansion in Water

Gradation	Total Expansion at End of 185 Days in Water, after Air Drying as Above, per cent	Fineness Modulus of Mix	W/C
Coarse	0.0385	5.69	0.80
Fine	0.0416	5.15	0.99

The tests at the University of California are especially valuable in that the exposure conditions with respect to humidity and temperature have been kept constant. On the other hand the exposures at Purdue have been those of the natural surrounding atmosphere. Temperatures have been kept constant, at the time of observations, to within 5 or 10 per cent and the measurements corrected with reference to temperature observations on duplicate specimens.

A bulletin is now being published which will record the results of the entire program. In general our results agree with those at the University of California except that we find that a large per cent of fine material increases the amount of shrinkage as shown in Table I.

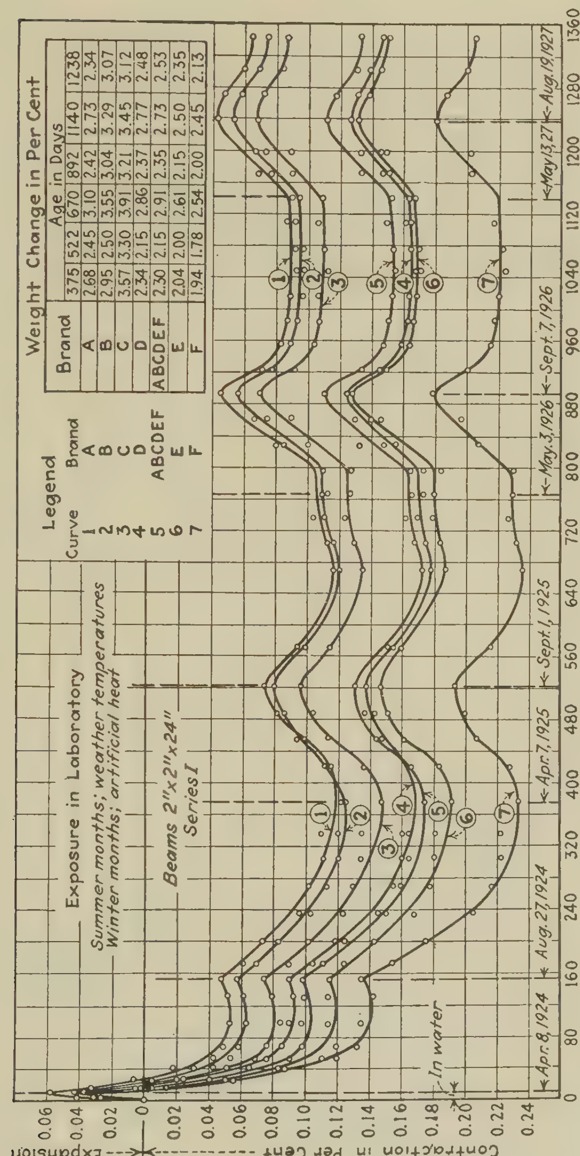

FIG. 1—CONTRACTION OF NEAT CEMENT BEAMS EXPOSED TO AIR OF LABORATORY.

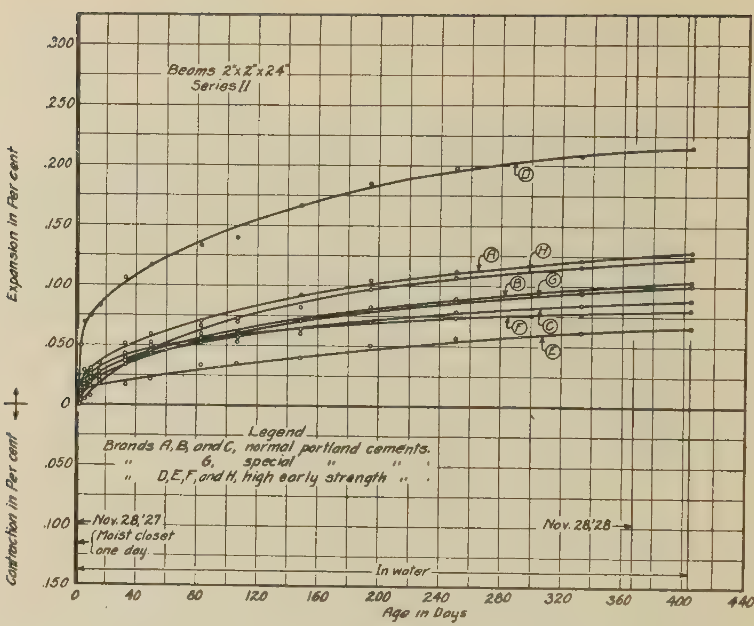

FIG. 2—EXPANSION OF NEAT CEMENT BEAMS IMMERSED IN WATER.

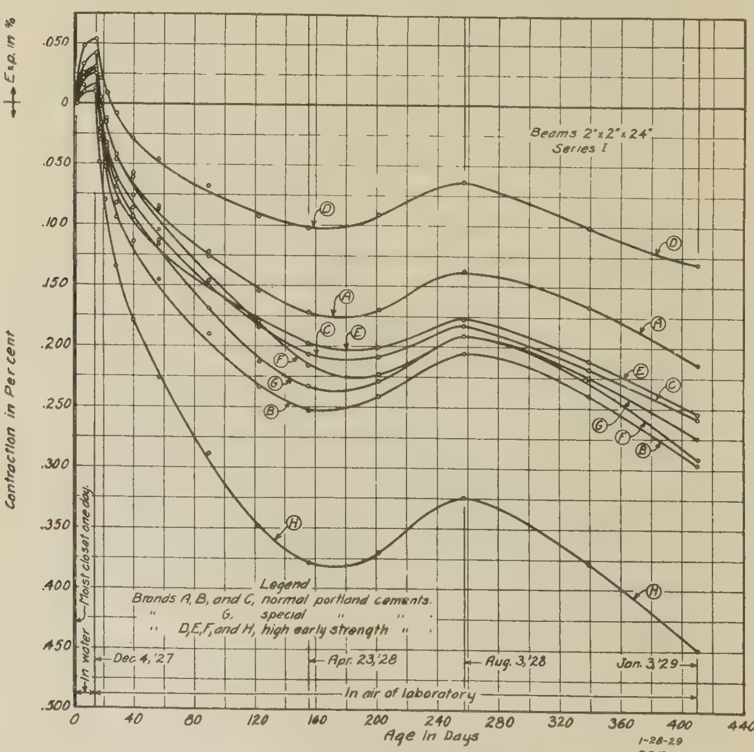

FIG. 3—CONTRACTION OF NEAT CEMENT BEAMS EXPOSED TO AIR OF LABORATORY.

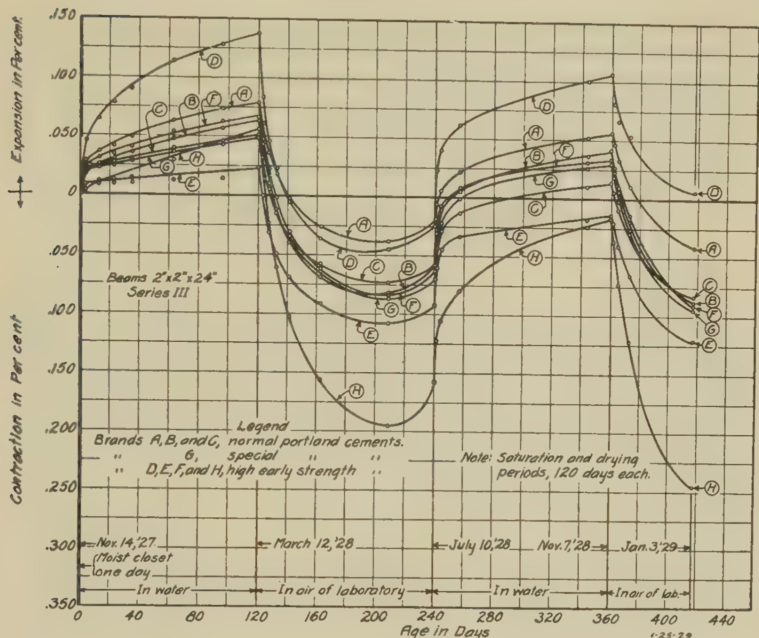

FIG. 4—EXPANSION AND CONTRACTION OF NEAT CEMENT BEAMS.

TABLE II—EXPANSION AND CONTRACTION OF NEAT CEMENT BEAMS.
(Continuously wet or continuously dry)

Kind of Cement	Contraction, 410 days in Air of Laboratory, Percentage (a)	Expansion, 410 days in Water, Percentage (b)	Range ($a+b$)	Range (Rating)
Normal Portland, A, B, C	−0.2560	+0.1068	0.3628	100
High Early Strength, E, F, H	−0.3329	+0.0903	0.4232	116
Lumnite, D	−0.1317	+0.2143	0.3460	95

TABLE III—EXPANSION AND CONTRACTION OF NEAT CEMENT BEAMS.
(Intermittently wet and dry, 120 day periods)

Kind of Cement	Contraction, 120 days in Air of Laboratory		Expansion, 120 days in Water		Difference ($a-b$)	Rating ($a-b$)	Sum ($a+b$)	Rating ($a+b$)
	Percentage (a)	Rating	Percentage (b)	Rating				
Normal Portland, A, B, C	−0.1297	100	+0.1017	100	−0.0280	100	0.2314	100
High Early Strength, E, F, H	−0.1779	137	+0.1309	128	−0.0470	168	0.3088	134
Lumnite, D	−0.1839	141	+0.1523	150	−0.0316	113	0.3362	146

The four accompanying diagrams exhibit some of the results obtained at Purdue. Figs. 2, 3, and 4 show the expansion and contraction of normal and high early strength portland cement together with Lumnite, and a brand called Quikard. The results are summarized and compared with the strength tests in Tables II, III, and IV.

TABLE IV—STRENGTH OF STANDARD CEMENT MORTAR.

Kind of Cement	Breaking Strength, lb. per sq. in. 1:3 std. Mortar							
	Compression				Tension			
	3 days		28 days		3 days		28 days	
	Test	Rating	Test	Rating	Test	Ratng	Test	Rating
Normal Portland, A, B, C	711	100	1995	100	179	100	369	100
High Early Strength, E, F, H	2006	282	3583	180	289	161	388	105
Lumnite, D	4225	594	4360	218	395	223	410	111

It appears that increased strength is attended with increased volume-change due to moisture-change.

Comparison of Methods of Determining Moisture in Sands

By William R. Johnson*

Purpose

In applying the water-cement ratio method of proportioning concrete to actual job conditions there is need for a quick, accurate method of determining the moisture content in the sand. A number of methods are now being used with more or less success. The basic principles of these methods, however, vary considerably. They include drying to a constant weight, measurement of the displacement in a liquid, the change in electrical resistance due to moisture, and the change in specific gravity of a testing liquid. All of these methods are not equally satisfactory. Some of them are not accurate, others either require too many operations and take too long to complete, or require too much apparatus for greatest usefulness under field conditions. This investigation was undertaken with a view to studying the advantages and disadvantages of some of these methods.

Methods Compared

The following methods of determining moisture in sand were compared by testing in turn three samples of sand each with 4 percentages of moisture:

(1) Electric resistance moisture meter;
(2) Drying to constant weight in oven;
(3) Drying to constant weight with denatured alcohol;
(4) Displacement method using cylindrical container;
(5) Displacement method using A.S.T.M. flask[1];
(6) Specific gravity method using salt solution Hydrometer.

Of the methods now in use probably the most common is that in which a definite weight of damp sand is dried to a constant weight and the difference in weight between the wet and dry sample is used in determining the actual moisture in the sand, including that absorbed by the aggregate. The drying by driving off the water from the damp sand by burning alcohol is a variation of this method.

The displacement method consists essentially in finding the difference in amount of water displaced by a damp sample and a dry sample of the same weight. Two of the methods tried are of this type. The method at

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[1] A.S.T.M. Standard C 70–28 T.

first appears more simple than the drying methods, but there is a greater number of separate manipulations required in making the determination, and the chance of error is increased with each.

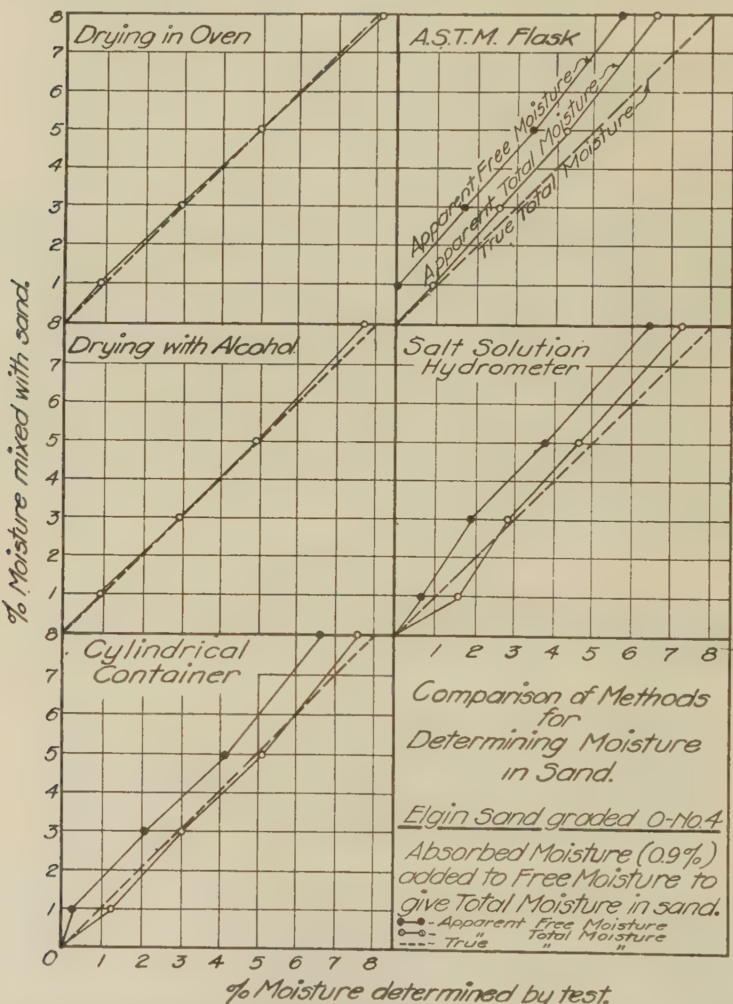

FIG. 1—COMPARISON OF METHODS FOR DETERMINING MOISTURE IN SAND GRADED 0-No. 4.

The specific gravity method is based on determining the specific gravity of a concentrated salt solution at a temperature of 70 deg. F. to which the damp aggregate has been added. The salt solution is diluted in proportion to the moisture content of the aggregate. Here

again the number of separate manipulations required in making a determination increases the chance of error.

The moisture meter or electrical method is for use primarily in determining the relative moisture contents of fine sands used in foundry work.

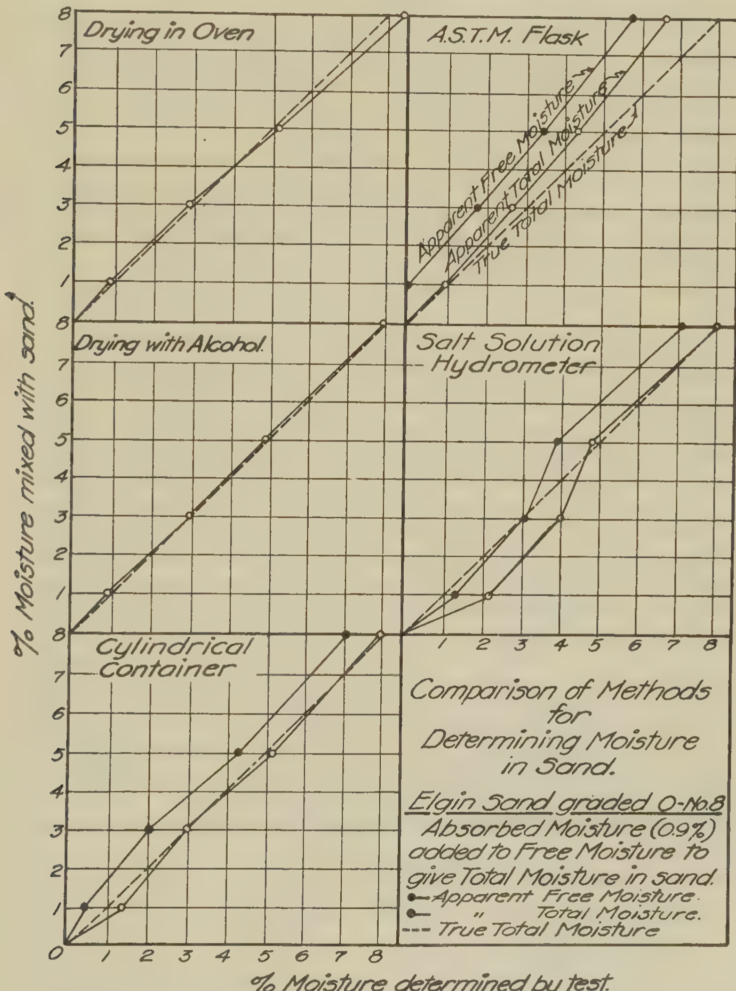

FIG. 2—COMPARISON OF METHODS FOR DETERMINING MOISTURE IN SAND GRADED 0-No. 8.

The principle upon which this meter works is the difference in resistance offered to an electric current by varying percentages of moisture in sand; the greater the moisture content, the less the resistance. In using

this method, it was found that a change in the grading of the sand acted the same as a change in moisture content and thus limited the accuracy of the method.

PROCEDURE

Preliminary Tests—In determining the moisture content of a sand when the cylindrical container or the A.S.T.M. flask was used, it was necessary to determine the displacement of the sand when dried to constant weight. These determinations were made preliminary to the tests made on the damp samples. The specifications of the A.S.T.M. (C 70–28 T) specify a surface dry sample in place of a sample dried to constant weight. It was found, however, that some trouble was encountered in determining the exact point at which a sample was surface dry, but still containing the water of absorption. A sample dried to constant weight was, therefore, used in place of the surface dry sample and its displacement read immediately, or before an appreciable absorption had taken place.

A quantity of concentrated salt (NaCl) solution was made up for use with the hydrometer tests. The tests were made on three prepared samples of Elgin sand graded 0–No. 4, 0–No. 8, and 0–No. 14. A 15-lb. sample of each grading, dried to constant weight, was placed in a pan and a definite percentage of water by weight of the sand was added and thoroughly mixed by kneading with the hands. This sample was used for test with each of the methods. Four different moisture percentages, 1, 3, 5, and 8, were used with each sand sample.

Sampling—After the entire sample had been used with the "Electric Resistance Meter Method" it was weighed out into smaller samples which were used in the following tests: Cylindrical Container, A.S.T.M. Flask, Hydrometer, and Drying to Constant Weight in Oven.

One operator was assigned to each of the above four methods so that all tests could be made simultaneously from the same large sample. When making duplicate tests, operators were changed.

The tests made by method of "Drying to Constant Weight with Denatured Alcohol" were not made from the same large samples used with the other 5 methods, but from separate samples.

The apparatus required and method of making each type of test is given below.

(1) *Electric Resistance Moisture Meter.*

Apparatus:

Moisture Meter

⅕-cu. ft. measure

Method—The entire sample of damp sand was placed in a ⅕-cu. ft. measure, the inside of which had been slightly dampened with a wet cloth to prevent any appreciable loss of moisture from the sand. The moisture meter was then inserted into the sand according to the manufacturer's directions and the indicated or relative moisture read on the meter. Three readings were taken for each condition.

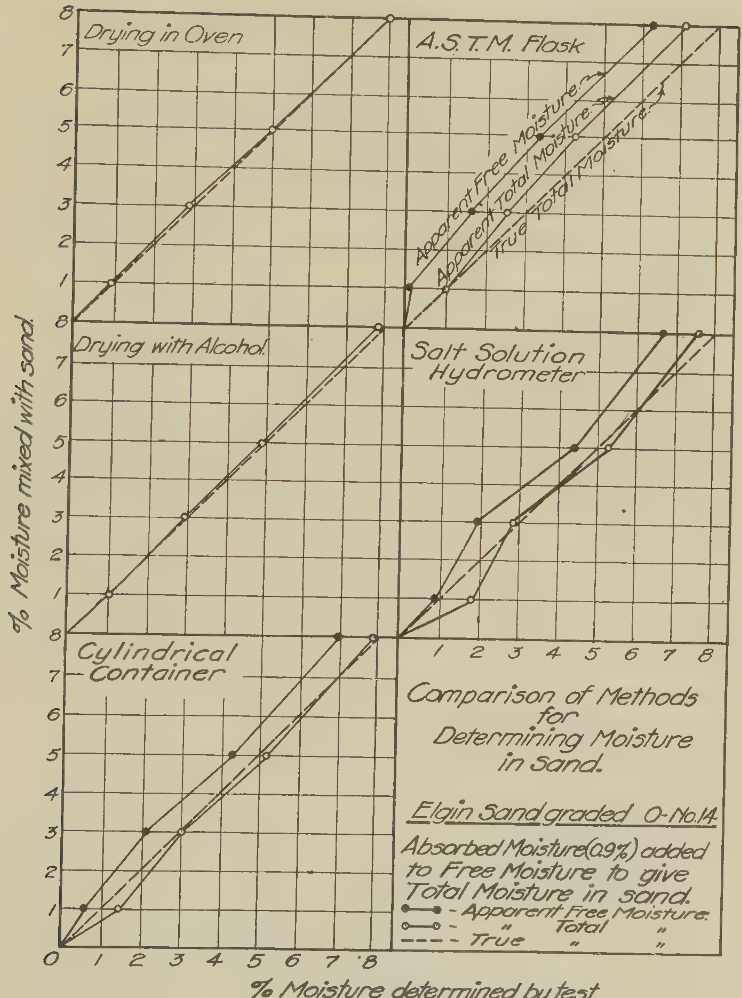

FIG. 3—COMPARISON OF METHODS FOR DETERMINING MOISTURE IN SAND GRADED 0-No. 14.

(2) *Drying to Constant Weight in Oven.*

Apparatus·

Balance sensitive to 1 gr.

Small pan

Drying oven

Method—A 1000-gr. sample of damp sand was placed in a pan. The

pan was placed in an oven, dried for 30 min., and then weighed. The total percentage of moisture was then calculated as follows:

$$p = 100 \frac{W - W'}{W'}$$

where p = percentage of moisture by weight of dry sample including absorbed moisture.
W = weight of damp sample.
W' = weight of dry sample.

3. *Drying to Constant Weight with Denatured Alcohol.*
Apparatus
Balance sensitive to 1 gr.
Small pan
Metal stirring rod about 15 in. long
½-pint cup
Denatured or wood alcohol

Method—500 gr. of damp sand were placed in the pan. One-third cupful of alcohol was poured over the sand; the mixture was stirred with the rod and then spread in a thin layer over the bottom of the pan. The alcohol was ignited and allowed to burn until consumed, the sand being stirred occasionally with the iron rod during burning. If the sand appeared damp after burning, additional alcohol was added and the burning process was then repeated in order to insure complete drying of the sample. After burning, the sand was allowed to cool for 2 or 3 min. and then weighed. The total percentage of moisture was calculated from the same formula as used with the "Drying to Constant Weight in Oven" method or $p = 100 \frac{W - W'}{W'}$

4. *Displacement Method Using Cylindrical Container.*
Apparatus:
Balance sensitive to 1 gr.
Cylindrical container as shown in Fig. 4 with gage glass and scale calibrated to read to 5 cc.
3 ft. of spring wire coiled at one end
8-in. funnel with bottom diameter about 1½ or 2 in.

Method—The cylindrical container was filled with water up to the zero mark on gage. The spring wire was inserted into the container with the coiled end resting on the bottom. A 2000-gr. sample of dry sand was poured into the container through the funnel and the wire gradually withdrawn to agitate the sand in order to remove entrained air. The volume of water displaced was read on the gage. The operation was then repeated, using same weight (2000 gr.) of the damp sand, whose moisture content was to be determined. The percentage of moisture was then calculated from the formula:

$$p = 100 \frac{D - C}{W - D}$$

where p = percentage of moisture by weight of dry sample exclusive of absorbed moisture:

D = weight of water displaced by damp sample of weight "W."

C = weight of water displaced by dry sample of weight "W."

W = weight of sample (dry, surface dry, or damp).

It was necessary to use a dry sample to establish the constant "C." This constant does not change as long as the specific gravity of the sand remains constant.

5. *Displacement Method Using A.S.T.M. Flask.*

Apparatus:

Balance sensitive to 1 gr.

A.S.T.M. flask as shown in Fig. 4.

6-in. funnel with bottom diameter about 1 in.

Method—The flask was filled with water to the 200-cc. mark. A 500-gr. sample of sand dried to constant weight was then poured into the flask through the funnel; the flask was well shaken and the water displaced noted on the flask. The operation was then repeated using the same weight (500 gr.) of damp sand. The percentage of free moisture was calculated from the same formula as used with the "Cylindrical Container" method or $p = \dfrac{D - C}{W - D}$

6. *Specific Gravity Method Using Salt Solution Hydrometer.*

Apparatus:

Balance sensitive to 1 gr.

2-quart jar or similar container.

Hydrometer as shown in Fig. 4 calibrated to read in gal. of water per 100 lb. of damp aggregate.

Metal stirring rod about 15 in. long.

Saturated solution of common salt (NaCl).

Method—One quart or 946 cc. of the saturated salt solution was placed in a 2-quart jar. A 1515-gr. sample of damp sand was poured into the jar and the mixture stirred vigorously for about 3 min. The hydrometer was then inserted and read. Three readings were taken for each condition. The percentage of moisture by weight of the damp sample was calculated as follows:

$$p = G \times C$$

where p = percentage of moisture by weight of damp sample:

G = hydrometer reading in gallons per 100 lb. of damp sand.

C = a constant, or the pounds of water in a gallon.

p was then converted to percentage of moisture by weight of the dry sand contained in the damp sample.

DISCUSSION OF TESTS

The results of the tests are given in Table 1 and in Fig. 1, 2, and 3. Fig. 4 is a photograph of four of the different types of apparatus used in the tests. No unusual or special apparatus was required with the two different methods of drying to constant weight.

Following is a brief discussion of each of the methods used:

(1) *Electric Resistance Moisture Meter*—This apparatus was manufactured for testing the relative moisture content of foundry sands, but

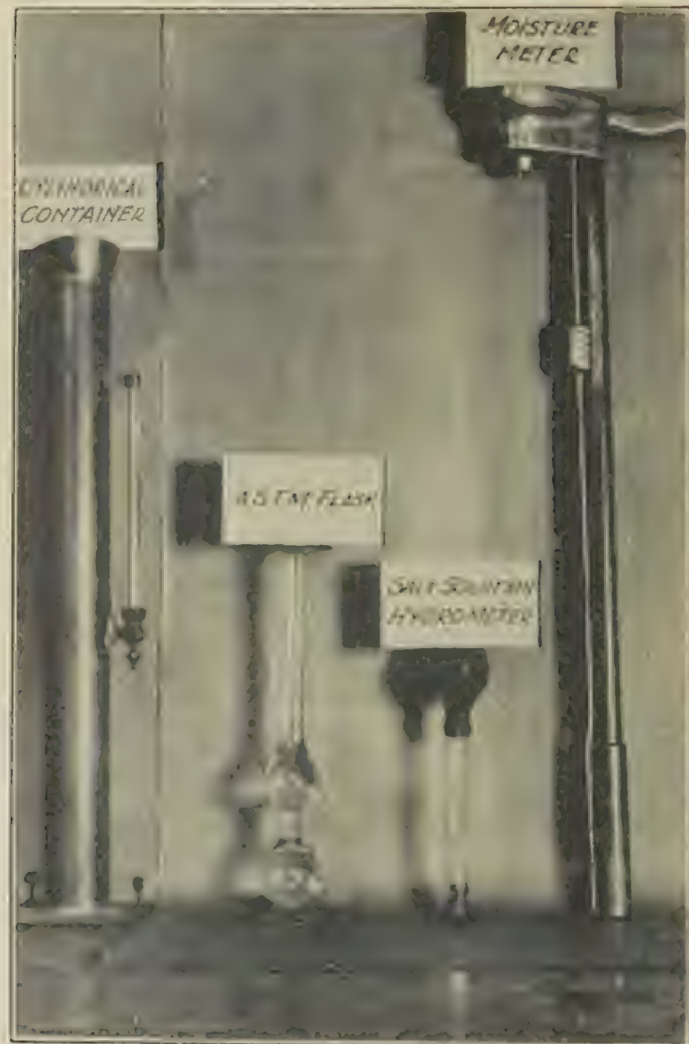

FIG. 4—DEVICES FOR DETERMINING MOISTURE CONTENT OF AGGREGATES.

was found to be not well suited for testing the courser sands used in concrete construction. While it was the quickest method used, the results were not accurate.

The readings were not given in per cent moisture obtained, but as relative moisture. It would, therefore, be necessary to plot curves showing the relation between the actual moisture content and the moisture meter reading. This relation would vary for each type and grading of sand.

(2) *Drying to Constant Weight in Oven*—This method was one of the two most accurate methods used. Its principal limitations are the necessity of a stove or drying device of some kind and the necessity of determining the absorption of an aggregate in order to calculate the free moisture contained in the sample.

TABLE 1—COMPARISON OF METHODS FOR DETERMINING MOISTURE IN SANDS

Determinations of moisture content of sands using 6 different methods.
Aggregate: Sands of 3 gradings from Elgin, Ill. The absorption of the sands at 15 min. was 0.9 per cent.
Moisture content: Surface moisture was the free water contained on the outside of the aggregate. Absorbed moisture was the water contained within the aggregate. Total moisture included the surface moisture plus the absorbed moisture.
Each value is the average of 2 determinations made by different operators.

Sample		Per Cent Moisture Determined by Test								
Grading	Per Cent Moisture Mixed with Sand	Drying to Constant Weight		Cylindrical Container		A.S.T.M. Flask		Hydrometer		Moisture Meter
		In Oven	With Alcohol*	Surface Moisture	Total Moisture	Surface Moisture	Total Moisture	Surface Moisture	Total Moisture	Relative Moisture
		Total Moisture								
0–No. 4	1.00	0.90	0.95	0.28	1.18	0.03	0.93	0.63	1.53	0.37
	3.00	2.96	2.92	2.09	2.99	1.68	2.58	1.92	2.82	2.00
	5.00	4.99	4.93	4.18	5.08	3.43	4.33	3.73	4.63	3.00
	8.00	8.14	7.69	6.63	7.53	5.70	6.60	6.41	7.31	4.50
0–No. 8	1.00	0.91	0.91	0.40	1.30	0.03	0.93	1.26	2.16	0.75
	3 00	2.86	2.98	2.00	2.90	1.58	2.48	3.06	3.96	3.00
	5.00	5.13	4.93	4.26	5.16	3.52	4.42	3.87	4.79	5.50
	8.00	8.47	7.96	7.04	7.94	6.38	7.28	7.11	8.01	4.50
0–No. 14	1.00	0.96	1.01	0.52	1.42	0.10	1.00	0.84	1.74	0.48
	3.00	2.88	2.91	2.05	2.95	1.60	2.50	1.90	2.80	2.00
	5.00	4.99	4.92	4.23	5.13	3.37	4.27	4.35	5.25	3.00
	8.00	8.00	7.80	6.95	7.85	6.26	7.16	6.65	7.55	4 00

* The sands used with this method were not taken from the same batches as were used with other five methods.

(3) *Drying to Constant Weight with Denatured Alcohol*—This method, from the standpoint of simplicity, accuracy, and speed, was probably the best of the six methods tried. Its one limitation was the necessity of determining the absorption of the aggregate in order to obtain the free moisture content.

(4) *Displacement Method Using Cylindrical Container*—The results obtained by this method were not as accurate as those obtained by the two methods of drying to constant weight, but ranked third in point of

accuracy. The advantage of this method over the drying out methods was that the percentage of moisture obtained was the free moisture, thereby eliminating a correction for absorption. Its advantage over the A.S.T.M. flask was the ability to read it more accurately, due to the scum not collecting in the water gage as was the case with the flask.

(5) *Displacement Method Using A.S.T.M. Flask*—As seen from Table 1 and Fig. 1, 2, and 3, the results obtained by this method were not as accurate as hoped for. The principal trouble encountered was the inability to read the water displacement accurately, due to the scum which collected on the top of the water in the neck of the flask. In order that comparable tests could be made, the flask was read at the line of demarcation between the water and scum. The wet sands collected more scum than the dryer ones; this probably explains the greater inaccuracies obtained with the wetter sands.

(6) *Specific Gravity Method Using Salt Solution Hydrometer*—This method gave the most inaccurate results of the five (not including moisture meter) compared. The hydrometer was not sensitive enough to be read closely. It is quite possible that by reducing the range and increasing the length of the instrument it could be made to read more accurately. The maximum range of the hydrometer used was from 0 to 3.5 gal. per 100 lb. of wet sand. This could easily be reduced to 2 gal. of water per 100 lb. of aggregate without sacrificing its usefulness for concrete sands.

It was necessary to check the density of the salt solution at frequent intervals in order to maintain a solution which gave the required density.

CONCLUSIONS

A comparison of all methods used indicated that the best results were obtained with those methods requiring the simplest manipulation. For example, the two drying methods gave the best results. The manipulation consisted in weighing before and after drying. The two displacement methods (Cylindrical Container and A.S.T.M. Flask) ranked next in point of accuracy. The manipulation consisted of weighing the samples and also in determining the displacement of water. The hydrometer method, which ranked last in point of accuracy, required weighing the sample, measuring a definite quantity of salt solution, and in reading the hydrometer accurately.

In developing further methods for determination of moisture in aggregates, effort should be made to keep the method as simple as possible in order to eliminate the pyramiding of errors which is bound to occur in a method using involved manipulations.

C. M. CHAPMAN -I would like to ask a question before I discuss this paper. The third method in the list is called "drying from constant weight with denatured alcohol." In using this method, what steps, if any, were taken to insure drying to constant weight? Mr. Chapman.

F. R. McMILLAN (who presented Mr. Johnson's paper)—The same as in the ordinary methods of drying—re-weighing. Mr. McMillan.

C. M. CHAPMAN—In Table I there is a constant difference of exactly 0.9 per cent between the values of surface moisture and the total moisture, where these two quantities are given side by side. In the heading over the table is the sentence, "The absorption of sands in fifteen minutes is 0.9 per cent." How is that absorption of 0.9 per cent, as given in the heading of Table 1, determined? Mr. Chapman.

F. R. McMILLAN—The method we use is a modification of one described by A. S. Roe in a paper, "Apparent Specific Gravity of Non-Homogeneous Fine Aggregate" (*Proc.* A.S.T.M., 1917, p. 257). It is based on the comparison of the displacement of the dried sample with the displacement of a similar sample previously soaked in kerosene. Mr. McMillan.

C. M. CHAPMAN—Was any investigation made to determine the extent of the inaccuracy due to scum, by allowing it to stand until the water cleared up under the scum? Mr. Chapman.

F. R. McMILLAN—The degree of inaccuracy is indicated by the results shown later. Mr. McMillan.

C. M. CHAPMAN—The paper compares six methods. Two of them are put aside as not suitable or sufficiently accurate. Of the other four, the two drying methods are said to be quite accurate for the determination of total moisture, and the other two, the displacement methods, are rated as less accurate for total moisture, with the implication, at least, that one of them might be used for determining surface moisture, but that the A.S.T.M. method is less accurate on account of scum. Mr. Chapman.

One method is selected as "probably the best of the six from the standpoint of *simplicity*, *accuracy* and *speed*. By "speed" I assume is meant time required for making a test.

Now I am not arguing against the alcohol burning test. I have used it many times, and with certain precautions it is useful. But one necessary precaution is not mentioned in the paper. The test is named in the list as "Drying to *constant weight* with denatured alcohol." But nothing is said about how one is to know that the sample is dried to constant weight. The first burn-off of alcohol may take off all the surface moisture and leave the sample apparently dry but with all of the absorbed moisture

still in it. I have burned off samples containing just about the maximum amount of moisture that one burning would remove and the flame would go out just as the sand lost its moist appearance. Apparently the tests reported in the paper did not come out that way, since results check so closely with oven-dried results, although there is one case—that of 8 per cent moisture in zero to 4 sand where there is a difference of 0.45—almost ½ of 1 per cent between the alcohol drying and oven drying. Since this case of greatest variation occurred with the wettest sand, it may perhaps have been caused as I have just described. To be sure of drying to constant weight (as the name of the method implies), the sample should be burned until it appears dry, then weighed, burned again and again weighed. This added procedure detracts somewhat from the "simplicity" of this method which is mentioned in the paper as one of its advantages.

The next matter is that of scum or froth which sometime is present on the top of some fluids. It is said to detract from the accuracy of the reading, but to an extent not disclosed in the paper. Instead of detracting from the accuracy of readings of the A.S.T.M. flask, the presence of froth really increases the accuracy. You all know the tendency of water in a tube to take on a saucer-like surface called the meniscus. Now when the meniscus is present there may be a doubt in the mind of the operator whether to read the top or the bottom of the meniscus and this might lead to some degree of inaccuracy. But when froth is present *there is no meniscus.* The top of the liquid is flat and the reading is not only easier to make but it is more accurate when made. The presence of froth is actually an advantage.

The question might be raised as to whether the water in the froth might, when it settled out, change the reading. Tests with very fine wet foundry sand which produced about an inch of froth have shown that the amount of water in the froth would not, when settled out after long standing, make the least change in the reading. This test has been made repeatedly. I recommend that Mr. Johnson make the test.

Another advantage given the alcohol burning test is that of speed. The time required to make this test depends partly upon the amount of moisture in the sand, that is, the number of burnings required. When but one burning is performed and 2 or 3 minutes allowed for cooling, I have made this test in 11 minutes. When two burnings are made, the best I have seen timed is 15 minutes, but if constant weight is to be insured by an extra burning and a third weighing, about 20 minutes is fair speed. That is not bad—not a slow test, compared, for example, with 30 minutes in a drying oven, but with the A.S.T.M. flask 5 minutes is ample time to make the test, determine the results and record them. I have made 10 complete separate tests, cleaned the flask, determined and recorded results in 27 minutes, or a little over 2½ minutes for each test. The alcohol test may be speedy but the A.S.T.M. test is speedier.

Now we come to accuracy. The surface moisture being the result desired, the alcohol method admittedly gives a result which is out by the amount of the absorbed water evaporated and the A.S.T.M method gave

results which were not as accurate as hoped for. I do not attempt to account for the inaccurate results obtained with the A.S.T.M flask. They are not borne out by many results of tests made by others. Since reading this paper a few days ago, I made a few tests following somewhat the methods and percentages of water reported therein. The results obtained with the A.S.T.M. flask were interpreted by means of the chart which accompanies the instrument. A siliceous sand—common about New York and called Cow Bay sand—was used. Two sizes, one quite fine which passed a No. 14 sieve and the other quite coarse which passed a No. 4 sieve. The sand was "room dry" to begin with, having been stored in a cellar for a long period. Weighed quantities of water amounting to 3, 5 and 8 per cent of the resulting mixture were added. Samples of this sand oven dried gave losses of 0.4 and 0.5 per cent. Dried with alcohol, the loss was the same. Moistened so that the resulting damp sand would contain 3 per cent more moisture than it had before wetting and testing in the A.S.T.M flask, the results were 2½, 3 and 2½ per cent.

With 5 per cent of moisture added, the flask showed 4.5 per cent of *surface* moisture in each of three tests. Oven drying showed 5.5 per cent, indicating a total absorption of 1 per cent, or practically the same absorption as the sand reported in the paper. When moisture was added up to 8 per cent, two tests with the flask gave 7.5 and 7.75 per cent.

Boosting the water to 10 per cent, which is pretty wet sand, the flask showed 10 per cent and 10 per cent on two separate tests. Burned twice with alcohol, it showed 9.6 per cent and after a third burning 10 per cent.

With the coarse sand, the results were of very much the same degree of concordance.

Five per cent samples showed 4⅝ per cent, while alcohol burning gave 5 per cent and a second burning gave 5.4 per cent on the same sample.

On an 8 per cent sample the flask gave 7.75 per cent and burning gave 6.6 per cent on first burning and 8 per cent on second burning.

You have noted that the flask gave results usually about ½ of 1 per cent low, which is just about what it should give, since this sand would take up—absorb—about ½ of 1 per cent more moisture than it already had in it under the conditions of storage.

On the score of accuracy it is probable that the A.S.T.M. flask yields to the drying methods, but since surface moisture only must be determined, I am of the opinion that if the A.S.T.M. method is followed—you note that the method of operation used in the tests reported was not the A.S.T.M method—the accuracy is quite satisfactory.

There is one more question to be discussed—that is, when is a sand surface dry? The paper mentions under "Procedure," that some trouble was encountered in determining the exact point at which a sample is surface dry, but still containing the water of absorption.

Whatever method is used, it is necessary in order to use the water-cement ratio method of proportioning, to distinguish between surface moisture and absorbed moisture. The determination of the absorbed moisture is therefore necessary when the drying method is used. The

paper states in the heading of Table I, "The absorption of the sands at 15 min. was 0.9 per cent," but does not state how this was determined.

With almost any sand or fine aggregate except a very clean white material, there is a noticeable change in color when the surface moisture has evaporated. This is a very sensitive indicator. One cubic centimeter of water added to a 500-gr. sample of sand which has been dried until its color has just changed will restore the damp color in three minutes. That color will disappear again if spread out in moderately dry air, and a centimeter of water, 15 drops, will restore that color. Accuracy within such a limit as that, I maintain, is close enough for all practical purposes.

When very clean white materials, such as Ottawa sand or screenings from white rock, are used, they can readily be discolored so as to be sensitive to this color test by the addition of relatively minute quantities of coloring pigment such as ochre or red iron oxide. Half a gram of color added to a sample is enough and will not affect any reading to a noticeable degree.

In conclusion, the A.S.T.M. flask is simple to operate by unskilled hands, or at least by anyone having skill sufficient to be intrusted with the making of any test. It takes less than half the time required by a drying test. There are no mathematical calculations involved. Its accuracy in determining surface moisture is quite satisfactory.

Mr. Mauro. F. Mauro—The interest an engineer has in moisture in sand is related to the influence which the moisture existing in the sand may have on the strength of the concrete or in determining the amount of water to determine the strength of the concrete. Therefore it will be desirable for the experimenters who have shown very great interest in the scientific results of their work, to give to us a limit whereby we can determine how accurate our tests need be in order to be of practical utility in determining the additional amount of water which is to be used. In other words, we need their valuable assistance in giving to us information as to the manner in which we can determine, in a gross way, the amount of moisture existing in sand that occurs in field work.

Mr. Johnson. W. R. Johnson—Mr. Chapman has criticized the alcohol method from the viewpoint of the number of burnings required to dry a sample to constant weight. In the tests recorded, but one burning was required for sands containing 1, 3 and 5 per cent of moisture, while two burnings were used for the sands containing 8 per cent of moisture. It was found, however, that by increasing the amount of alcohol used from ⅓ to ½ cupful all the moisture could be driven off by one burning. An important point to be remembered in making the alcohol test is that constant stirring is necessary in the later stage of the burning in order that the flame may not become extinguished before all of the alcohol has burned off.

Mr. Chapman has also discussed at some length the scum which the writer has referred to in the discussion of the A.S.T.M. flask. He speaks of a determination made with a very fine wet foundry sand which produced about an inch of scum or froth. He says that the amount of water in the froth did not, when settled out after long standing, make

the least change in the reading. At the time the moisture tests forming the basis of the paper were being made several determinations of this type were carried out in an effort to determine if possible some method of overcoming this objection. Since receiving a copy of Mr. Chapman's discussion the writer has made further tests along this same line. The results show that an inch of froth, which represents about 11 cc. of total volume, will produce about 2 cc. of water after settling for 2 hours. This additional water will add about 0.7 per cent of moisture to the original determination. Although the sand used in this test was a washed sand of calcareous origin, it may have been somewhat dirtier and more scum producing than that used by Mr. Chapman in his tests.

George A. Smith* (*By Letter*)—In concluding his paper, Mr. Johnson points out that new methods should be as simple as possible. A field laboratory should not be encumbered with complicated, delicate apparatus. The simplest equipment is the best, provided that the results obtained are reliable within desired limits. **Mr. Smith.**

A simple and satisfactory method for determining the moisture in sands and coarse aggregates has been used in our laboratory, which may be of interest to members of the Institute. The only equipment necessary other than the balance found in every field laboratory is a bucket of water and a quart tin can fitted with a bail and a wire or cord by which the can may be suspended from the balance and immersed in the bucket of water. A small can containing sand or lead counterweight to balance the weight of the can when immersed will expedite the tests materially.

The method used in making the moisture determination is as follows: The weight of 1000 gr. of dry aggregate is determined while immersed in water, care being taken to remove entrained air by stirring with a small rod or stick. This weight is a function of the apparent specific gravity of the aggregate. This may be used as a constant which will not change appreciably so long as the source of the material is unchanged. To determine the moisture in the aggregate weigh out 1200 gr. of moist sample. From this not over 1000 gr. is inundated and stirred in the immersion can to remove the air. The can is then suspended from the balance and immersed in the bucket of water. By means of a small scoop, piece of bent tin or other convenient implement, small amounts of the remaining moist aggregate are allowed to fall into the can until the balance set at the previously determined weight balances. The difference between the weight of the *remaining* moist aggregate and 200 gr. represents the moisture in parts per 1000 based on the weight of the dry aggregate.

The above is based on the assumption that the aggregate is non-absorbent. If, however, it is absorbent, the initial weight of the immersed dry material should not be taken until after the balance remains in equilibrium for some time, indicating that the aggregate has absorbed nearly as much moisture as it will normally carry as absorbed water when supplied in a damp condition. The procedure is then as with the non-absorbent aggregate and the absorbed water factor is automatically cared for.

* The Celite Corporation, Los Angeles, Calif.

There are several features about this method that recommend its use. In the first place, a large amount of the entrained air is removed which in some displacement methods makes the apparent free moisture greater than the real. The apparatus is exceedingly simple and easy to rig up in any laboratory. The free moisture factor can be determined to within two or three parts per thousand of dry material. The method also may very conveniently be used for determining the rate of absorption of absorbent materials furnished in a dry condition.

Mr. Philip. PATRICK PHILIP* (*By Letter*).—The Public Works Department of British Columbia has devised a very simple method of determining the moisture in sands, a method possessing some advantages of its own and requiring no special apparatus other than a simple narrow cylindrical graduate glass and some water.

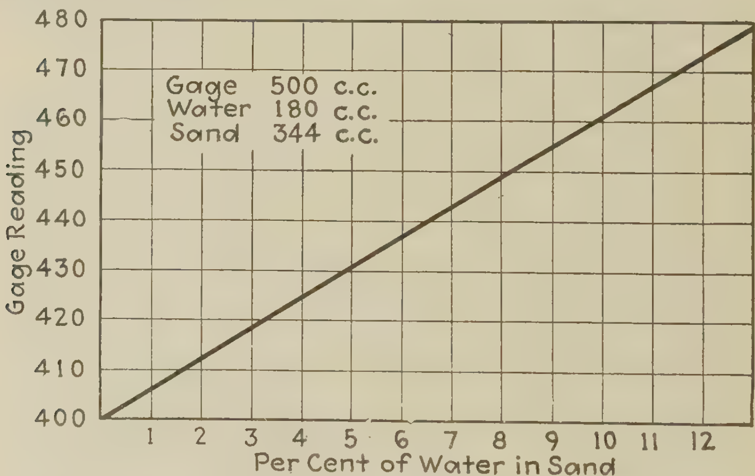

FIG. 1.—FIELD POCKET CARD GIVING CALIBRATION OF CYLINDRICAL GRADUATE—THAT IS GAUGE READINGS AS AGAINST MOISTURE IN SAND.

Calibration of the cylinder for any given sand occupies less than thirty minutes and the actual field operation of determining moisture can be carried out in less than one minute.

For calibration, it is only necessary to make two or three readings for the purpose of constructing a simple diagram (see Fig. 1), the apparatus required for calibration being:

500 cc. (or 250 cc.) cylindrical glass graduate,
A wide-mouth funnel to fit cylinder,
A laboratory scale,
Sample of job sand, surface dried (5 lb.).

Calibration is as follows:

The cylinder is partly filled with water up to a chosen mark (say 180 cc.); any convenient quantity of dry sand is then introduced into

* Deputy Minister and Public Works Engineer, Province of British Columbia, Canada.

the cylinder so that all the sand is inundated and so that there still remains some free water; the height of sand (Example 344 cc.) and the height of water (called the gauge reading), (Ex. 400 cc.) are recorded. The contents of the cylinder are then emptied out.

The cylinder is again partly filled with water to the same mark (Ex. 180 cc.) as before and sand containing a known percentage of water (Ex. 10 per cent) is introduced until the sand reaches the same level in the cylinder as the dry sand reached in the first experiment (Ex. 344): the height of water (gauge reading) is again recorded (Ex. 460 cc.).

As a check the experiment may be repeated once or twice or, alternatively, damp sands containing other percentages of water may be tried.

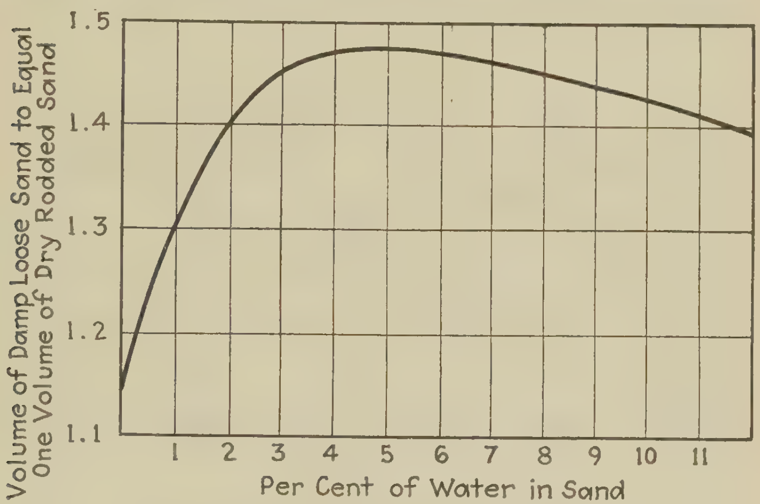

FIG. 2.—TYPICAL BULKING CURVE FOR SAND.

The results are plotted on a field card (Fig. 1), gauge readings vertical and percentage of water horizontal, the points are connected by a straight line and the calibration is now complete.

For field use, the inspector partly fills the cylinder with water to the same height as was used in calibration (Ex. 180), adds the damp sand to the fixed mark (Ex. 344) and notes the height of water in the cylinder (gauge reading). Consulting his pocket card, he reads off the percentage of water corresponding to the gauge reading and the test is complete.

The method may be varied by dispensing with the field pocket card and substituting a scale affixed to the cylinder marked:

Water level (Ex. 180).
Inundated sand level (Ec. 344).
Other levels corresponding to various percentages of moisture.

It will be noted that it is not necessary to weigh out the amount of sand which, in calibration, is introduced into the cylinder, the basic assumption being that a given weight of dry sand, when inundated, will always occupy practically a uniform volume. Scales are also completely dispensed with in the field tests. It will be clearly seen that increase in height of water in the cylinder represents the additional surface water contained in the damp sand introduced into the cylinder.

The accuracy of the method is dependent on the uniformity of: (a) the method of introducing the sand into the cylinder and (b) the method of reading the height of sand and water in the glass; but provided that the methods used in calibration are duplicated in the field, the errors are found to be less than one per cent, which is usually sufficiently accurate for field work.

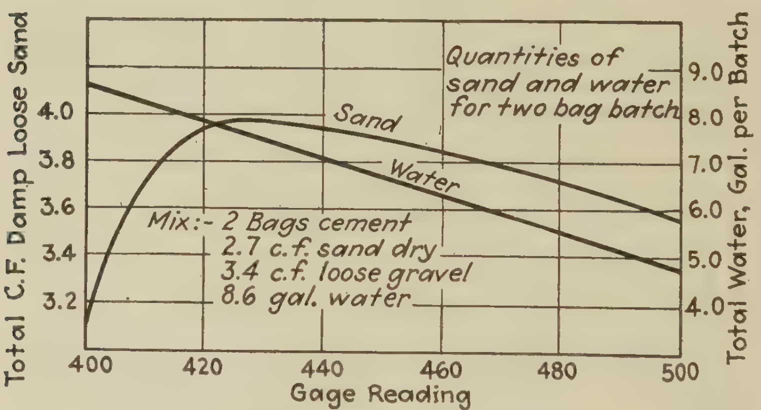

FIG. 3.—CHART GIVING TOTAL WATER AND TOTAL SAND FOR ANY PERCENTAGE OF WATER FOR A GIVEN MIX.

For the above reasons it is preferable that, during calibration, the operations and readings should be performed by the field inspector himself, whereby the personal equation is eliminated.

In introducing the sand into the cylinder, one of three methods may be used, viz: (1) loosely, (2) shaken, (3) rodded.

Any one of these methods gives satisfactory results, provided that the *same* method is used throughout.

In reading the height of water in the glass it is usual to read the bottom of the convex surface which is presented to the eye through the glass.

The Method Applied to Bulking of Sand.—The bulking of sand is a function of its moisture content. A large number of experiments in the investigation of this phenomenon with a view to providing a simple method for its field determination have been made.

For the sand in question, a bulking curve similar to Fig. 2 is made from actual experiment, such curves being all of similar shape. It is only necessary to obtain about four or five points to construct them. Correction must be made to adjust any difference between actual bulking as determined from job conditions and as determined in the laboratory.

As soon as the inspector has made his determination of moisture content, a glance at his bulking curve indicates the bulking corresponding to the percentage of water.

Fig. 3 is merely a further development giving at a glance, on one chart, the total water and total sand for any percentage of water, for a given mix.

A Scientific Trial Method for Designing Concrete Mixtures

By R. E. Robb*

For a number of years, the writer has been engaged in teaching scientific design of concrete mixtures to senior civil engineering classes. After repeated attempts to get them to grasp the methods commonly used, he was, with considerable reluctance, forced to the conclusion that the scientific method of design for concrete mixtures was too involved for general job use with the equipment now available except where a competent engineer could be employed continuously. This was further brought home to him when he was preparing for an extension class, consisting of engineers, architects, contractors, material men and foremen. It was simply unthinkable that in a period of 12 two-hour lessons these men, many of whom were unaccustomed to mathematical terms and calculations, would be able to find their own way through the maze of computations necessary for the mathematical design of concrete mixtures. Furthermore, it is freely admitted by the authors of this method of design that the charts and data on which it is based are absolutely correct only for certain aggregates and under certain conditions. It is freely admitted that these are valuable as a check, but it is also understood that they will not give the final answer in all cases. On the other hand, it seemed as though there must be some more scientific and at the same time more practical way than just the straight trial method of design, whereby we cut and try and try again until we think we have the most economical mixture. It was in the endeavor to arrive at this more scientific and yet practical method that the writer engaged in research during the past year.

The most basic and fundamental fact with which we have to deal in connection with concrete design is freely admitted to be the water-cement ratio, and as this is the most important factor, it was attacked first. As weighing of aggregate is, without doubt, the most scientific method of proportioning so far developed, a curve was first derived to show the relation between the amount of water and the per cent moisture on a dry aggregate basis in a given weight of aggregate. If we let x equal the weight of water in a sample of aggregate, and w equal the weight of the damp sample, then by definition $\frac{x}{w-x}$ equals the percentage of moisture. If w remains constant, say 100 lb., and various percentages

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be substituted in the equation, the value of x may be found, and from this the curve of Fig. 1 is plotted. If we know the percentage of moisture in an aggregate, we can, from this curve, determine how many pounds of water are contained in 100 lb. of the damp aggregate, and the proportion of the mixing water may be varied accordingly.

But although the modern trend is towards weighing aggregate, there are still a great many volumetric batchers in use, and it is probable that much concrete will be proportioned by volume for years to come,

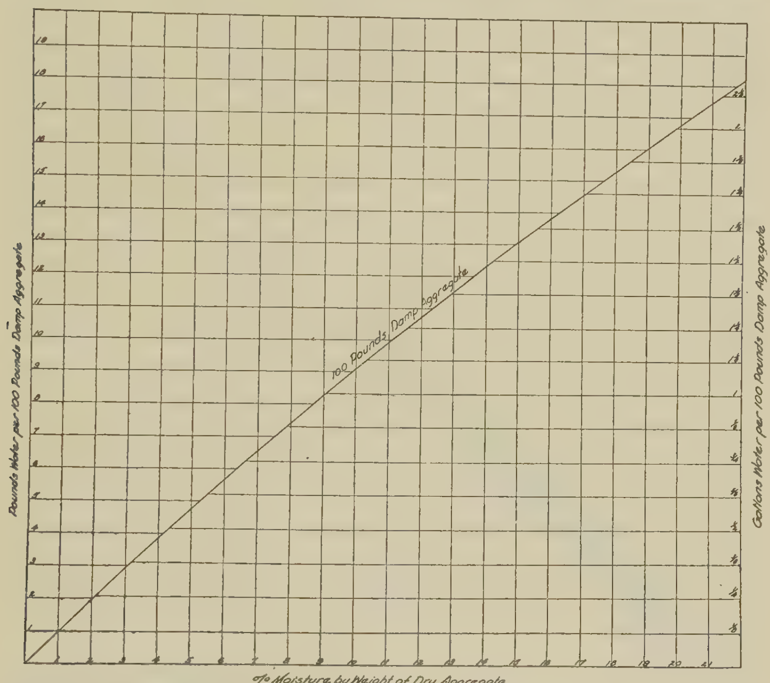

FIG. 1—RELATION BETWEEN AMOUNT OF WATER AND THE PER CENT MOISTURE BY WEIGHT OF DRY AGGREGATE.

especially on the smaller jobs. When this condition prevails, the problem of moisture determination resolves itself into determining how much water is actually contained in 1 cu. ft. of the material in a damp loose condition, as measured on the job. But the phenomenon of bulking enters to complicate matters. It seemed to the writer, however, that there must be some relation between the percentage of moisture and the actual amount of water in a cubic foot of the damp loose material. It is evident that there could be no mathematical relationship such as exists when the aggregate is weighed because of the difference in bulking due to the variation in grading of the materials. Accordingly, approx-

imately 50 samples of aggregate from widely separated locations were secured in an endeavor to determine whether or not a curve of general application might be determined. The procedure in making the determination was to start with 12 lb. of air-dried aggregate and a 1/10 cu. ft. measure. To the 12-lb. sample of aggregate, a known amount of water was added so as to give a definite percentage of moisture. This was then shoveled loosely into the measure, struck off, and the weight of damp aggregate in the measure determined. The water was added

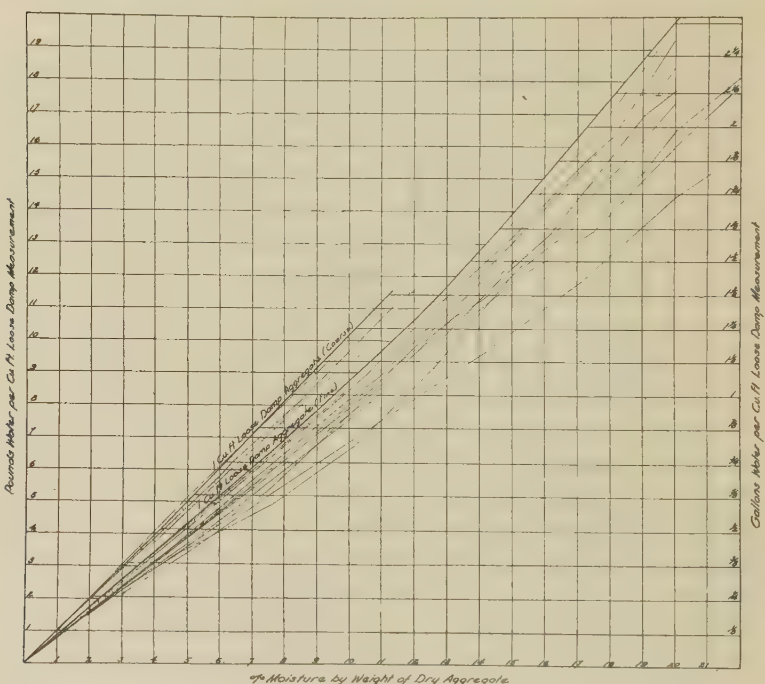

FIG. 2—RELATION BETWEEN PERCENTAGE OF MOISTURE AND AMOUNT OF WATER PER CU. FT. OF DAMP LOOSE MATERIAL.

in increments of 0.3 lb., so as to give an addition of 2½ per cent at each weighing. Knowing the percentage moisture, and the total weight of the aggregate in the 1/10 cu. ft. measure, the solution of the formula $\frac{x}{w-x}$ gave the amount of water actually contained in 1/10 cu. ft. of the damp loose material. Water was added until the point of saturation was reached, the data was tabulated, and the results plotted. These curves are shown in Fig. 2. It will be noted that the band in which these curves fall is comparatively narrow. The curves toward the

bottom of the band are, in every case, from sand which is ordinarily considered too fine for economical concrete. From a fineness-modulus of about 2.50 up to 3.25, there is a surprising similarity in the curves. The band is very narrow at 10 per cent, the variation being less than 1 lb. of water per cu. ft. These sands were obtained in North Carolina, Pennsylvania, Ohio, Wisconsin, Kentucky, Illinois, and Indiana, and from quite a variety of sources, pits, rivers, glacial deposits, etc. While it would be desirable to continue this study, it is felt that since the curves of these sands fell so close together, a general assumption is warranted. Accordingly, the curve of moisture content has been located as shown. It will be noted that this is on the upper and, therefore, the safe side of the band as respects the strength of the resulting concrete. Only a very coarse sand will go above this and even then it is very probable that the inaccuracies due to volumetric batching would be larger than those resulting from use of this curve. A similar curve is shown for gravels. Only a limited number of gravels were tested but as the curve goes higher for the coarser gravels which, of course, do not carry much moisture, it is felt that the gravel curve shown is conservative.

From these curves, if we know the percentage of moisture in an aggregate, we can determine within close limits the amount of water actually in a cubic foot of the damp loose material. It does not seem to make much difference how the material is loaded into the container. Tests were made using a 1-cu.-ft. measure, and 120 lb. of material, in which the material was first simply shoveled into the container as into a wheelbarrow, and then was dropped about 5 ft. into the measure, as when the material falls out of a hopper. There was a little more sand in the container when it was dropped 5 ft., but the additional amount of moisture was so small that it could be neglected in practice.

It is a very simple matter to determine these curves accurately on the job, for the particular aggregate which is being used. All that is necessary is 120 lb. of dry aggregate, a 1-cu.-ft. container, and a platform scale. Add known amounts of water to the aggregate, mix thoroughly, load into the container, strike off, weigh, and compute the amount of water. Increments of 3 lb. of water, or 2½ per cent, have been found convenient.

The next step in the solution was to find a simple rapid and easy method of determining the percentage of moisture in the aggregate. An investigation of all the instruments available for making this determination left the writer with the feeling that they were either inaccurate or too complicated for general job use. It was felt that the method of water displacement offered the most promise, and, using this method, a device has been prepared by which the amount of moisture in the aggregate is determined direct, without the use of any computations, charts, or other mathematical aids of any kind. After the apparatus is calibrated for the particular aggregate to be tested, it is only necessary to drop the damp aggregate into the container until a level bubble centers, and then read direct on a scale the amount of water carried by the aggre-

gate. This apparatus depends upon the specific gravity of the material for its action. It was primarily designed to determine the moisture content,

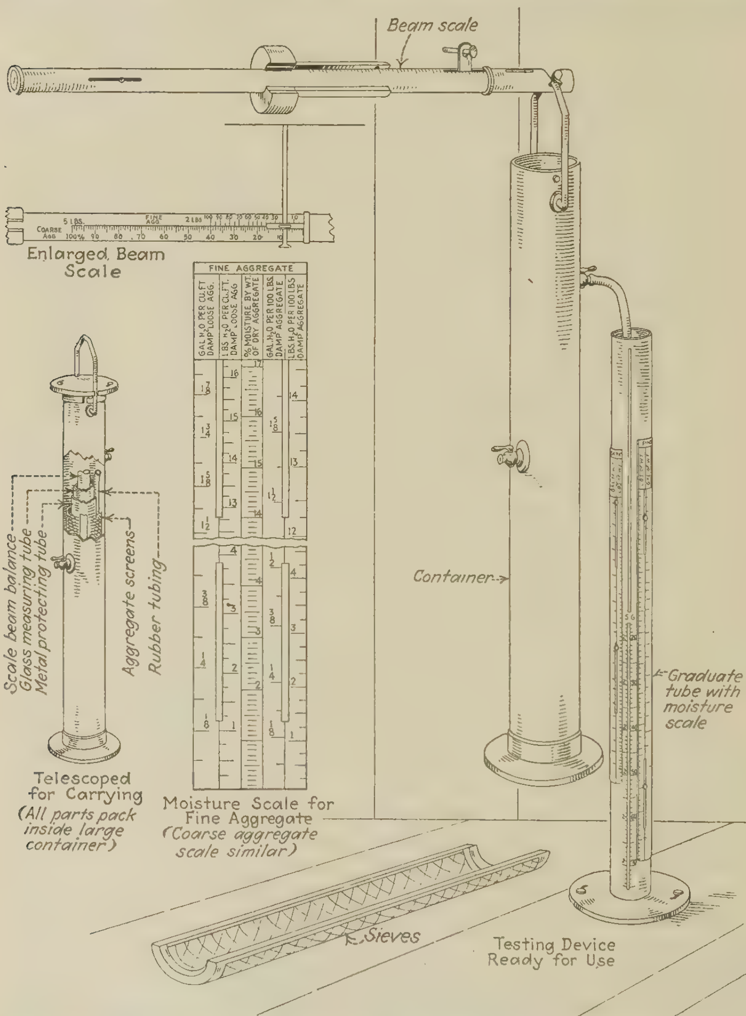

FIG. 3—DEVICE FOR DETERMINING PERCENTAGE OF MOISTURE IN AGGREGATE.

The apparatus which consists of four main parts—a beam scale, a container, a graduate tube and a set of sieves—may also be used to determine specific gravity, absorption of coarse aggregate, and to make colorimetric and sedimentation tests.

but with it may also be determined the specific gravity of the material, the absorption of coarse aggregate, and it may be used in the colorimetric and sedimentation tests. All of these determinations are made without

the use of any charts, tables, or computations. While it is an accurate, and extremely sensitive scientific instrument for the engineer to use it may be used with equally good results by the practical man who does not understand the principles on which it is based. Fig. 3 shows various views of the instrument and its parts.

It will be noted that the moisture scale is divided into 5 parts, labeled as follows: lb. of water per 100 lb. damp aggregate; gal. of water per 100 lb. damp aggregate; percentage moisture by weight based on weight of dry aggregate; lb. of water per cu. ft. of damp loose aggregate; gal. of water per cu. ft. of damp loose aggregate. These scales have been prepared from the curves shown in Figs. 1 and 2. The amount of moisture in the aggregate is determined by reading the height of the column of

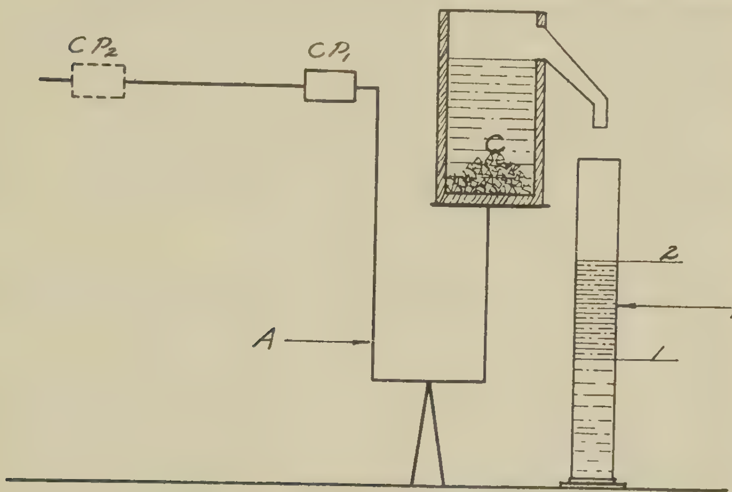

FIG. 4—SKETCH OF PRINCIPLE OF OPERATION OF DEVICE FOR MEASURING PERCENTAGE OF MOISTURE IN AGGREGATE.

water which is caught in the graduate on whichever scale is appropriate for the particular job. Thus, if the material is being weighed, it is desirable to know the amount of water per 100 lb. of damp material, while if it is being measured, the amount should be known per cu. ft. of damp losse material. The accuracy of the instrument depends entirely upon the specific gravity of the material being tested. Extensive tests have shown that the specific gravity of sands is practically the same. Coarse aggregates show a much wider variation in the specific gravity of samples from different locations, but if the source of material is constant, the specific gravity can be depended upon to remain constant within very close limits.

The fundamental principle of the instrument may be better understood from the diagram in Fig. 4. Container "C" has a spout with a well-defined weir, which accurately determines the volume of water,

provided the instrument is kept level. "B" is a graduate, which catches the overflow from "C". "A" is a platform scale on which "C" rests. The container "C" is filled with water, and the scale balanced by counterpoise in position CP_1. Then if 5 lb. of normal (saturated, surface dry) aggregate are placed in "C", it is evident that an amount of water will be discharged into "B" equal to the absolute volume of the 5 lb. of aggregate. The height of water in "B" should be noted, Point 1, and the counterpoise on the beam scale should be moved to the dotted position $C.P_2$, to exactly balance the weight of water and aggregate "C". "C" is now emptied, is refilled with water, and the moist aggregate to be tested poured into "C" until balance is again reached, the counterpoise being in the second position, $C.P_2$. If the specific gravity of the aggregate is constant, there must be exactly 5 lb. of aggregate in "C". It will be found, however, that the column of water in "B" is higher than in the first case, reaching point 2. The difference between these two columns of water, i.e., the volume 1–2, is the amount of water actually carried into the container "C" by the 5 lb. of aggregate tested.

The testing device, when used for moisture determination, is simply a modified form of the apparatus shown in Fig. 4. In order to reduce the size of the graduate without sacrificing accuracy the overflow weir has been raised to the point where a comparatively small amount of water is discharged by 5 lb. of normal aggregate.

The absorption of coarse aggregate may be determined with the same instrument, in a similar manner, and the specific gravity may be read direct on the scale at the end of the first operation described above.

We have thus a practical and rapid method of determining the amount of water actually carried by a unit quantity of fine and coarse aggregate. To keep the total amount of mixing water constant, it is only necessary to measure our aggregate accurately, either by volume or weight, and to deduct the amount of water in the aggregate from the mixing water.

The proper control of the water-cement ratio is the key to success in making quality concrete. From the contractor's point of view, however, quality is not the only consideration; economy must also be considered. The writer feels that the simplest way to think of concrete design is to consider a given amount of water-cement paste into which it is desired to mix as large an amount of aggregate as possible. The method explained above gives us a means for accurate control of the water-cement ratio. The problem of economy, then, resolves itself into using as much aggregate as possible in the given amount of water-cement paste.

It is felt that the simplest way to secure the desired results, viz., maximum yield for predetermined strength, is to hold the water-cement ratio carefully, and then, by trial batches, determine the proportions of the ingredients which will give the maximum yield with the required workability. The first step is to make a trial batch, using any arbitrary

proportions which it is thought will give usable concrete, say 1:2:4. The quantities of the materials used should be very carefully measured either by volume or weight and the yield noted in the receptacle, pre-

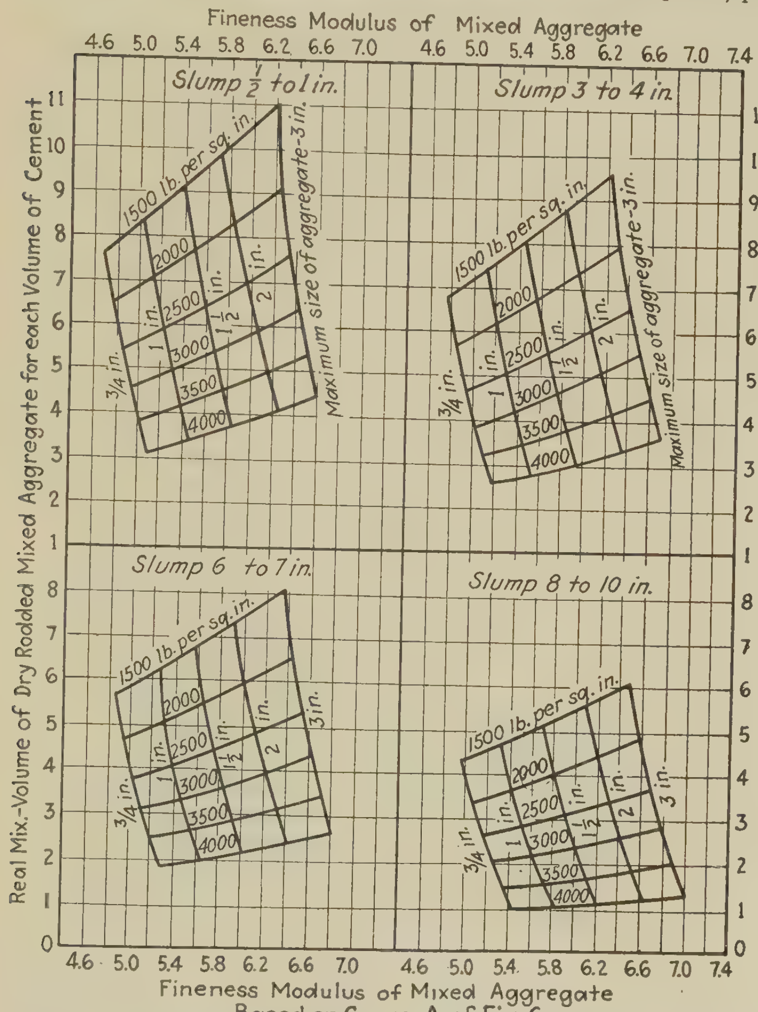

FIG. 5—FINENESS-MODULUS OF MIXED AGGREGATE.
From "The Design and Control of Concrete Mixtures," Portland Cement Association.

viously calibrated, into which the mixer is discharged. Careful examination of this batch will show how to proceed next in changing the proportions. A few trials, and the experienced concrete man will have

reached what he believes to be a good yield. The only thing for him to watch is that as he changes the quantity of aggregate he also changes the amount of mixing water added so as to keep the water-cement ratio constant.

After what is considered to be the best mix is secured, it should be checked both for fineness-modulus of mixed aggregate used and for

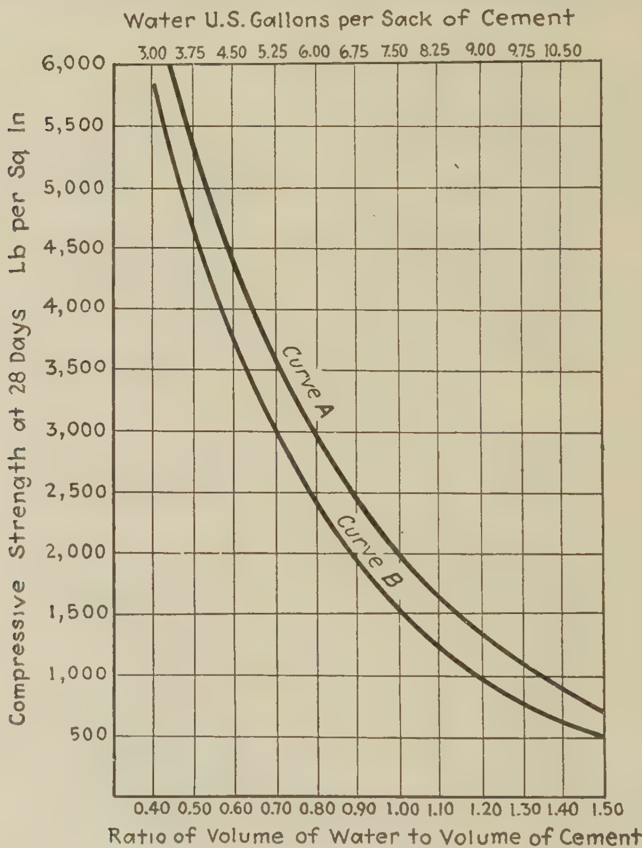

FIG. 6—ABRAMS' CURVES FOR WATER-CEMENT RATIOS.

water-cement ratio. To check the fineness-modulus, the aggregate should be charged into the mixer without the cement and water, and thoroughly mixed. A sample of this mixed aggregate is then dried and analyzed. This should be compared with the ideal fineness modulus for the given conditions, as found in the curves of Fig. 5, reproduced from "The Design and Control of Concrete Mixtures," prepared by the Portland Cement Association. If the fineness-modulus of the mix being

used falls below that shown in this chart, it is likely that more coarse material can be added, and the yield thereby increased. If it comes above that shown in the chart, care should be taken that the mix does not become too coarse for proper workability. The values from this chart are not absolute and will not hold for all conditions or for every aggregate. They are, however, a very considerable help in making scientific concrete. The water-cement ratio may be very easily checked by drying out a unit volume, say 1/10 cu. ft., of wet puddled concrete. This should be caught at the mixer, weighed, dried, and reweighed. The difference in weight will, of course, be the water driven off. If the absorption of the aggregate is deducted, the balance is the weight of water mixed with the cement in the sample. As the volume of the mix is known by means of the calibrated container, and the volume of the sample is known, the water-cement ratio as actually mixed may very easily be determined. The yield of each batch may be checked very easily by noting its height in the graduated receiving hopper. Any decided change in volume indicates a change in the water-cement ratio, in the quantities of aggregate charged or in the fineness-modulus. Either the quality or the cost has been affected.

The steps to be followed in making a scientific trial design may be summarized as follows:

(1) From Abrams' curve for water-cement ratios, Fig. 6, determine the water-cement ratio required.

(2) Determine the percentage of moisture in both fine and coarse aggregate.

(3) From curves of Fig. 1 or 2, determine actual amount of water in unit quantity of each damp aggregate.

(4) Make trial batch, compensating for moisture in aggregates, to obtain desired water-cement ratio.

(5) Change proportions until apparently ideal mix is secured.

(6) Note yield in graduated receiving hopper.

(7) Analyze mixed aggregate for fineness modulus, and compare with charts. Make any changes indicated.

(8) Check water-cement ratio by drying sample.

(9) Check yield of each batch by noting volume in graduated receiving hopper.

Mr. McCall.

H. C. McCall*—A method that we have been using is identical in principle with that described by Mr. Smith (p 275). We have a different way of applying it, and thinking that the detail may be of interest, I want to take just a moment to describe it.

The apparatus required consists of a balance of 3 kg. capacity, sensitive to the nearest gram and equipped with tare beam; also, two ordinary metal buckets, one a 10-qt. bucket, and the other a 1-qt. bucket. The 10-qt. bucket is filled about two-thirds full of water. The 1-qt. bucket is also filled with water and hung from the balance so the rim of the small bucket is just submerged in water in the larger bucket. Adjust weight on tare beam so that the apparatus is balanced.

Next determine the amount of sand which, when weighed submerged in small bucket, will just balance a given size weight, say 1000 gr. This will be 1600 to 1800 gr. of damp sand, depending on amount of moisture present and value of apparent specific gravity of sand. In practice, after apparatus is balanced the small bucket is removed and partly emptied. About 1500 gr. of sand is added through a tube (about 2 in. diameter and 12 in. long with funnel top), placing bottom end of tube on bottom of bucket and raising slowly as sand is added. This reduces the tendency of fines to remain in suspension. The sample should be poured in slowly to avoid entrained air. The small bucket is then hung on the balance, the water level adjusted so the small bucket is just submerged, and enough sand added through the tube to balance the 1000-gr. weight.

The sand in the small bucket is then dried, weighed, and the weight of absorbed water determined. The sum of these two weights is a constant which is used later to determine amount of free moisture in any other sample.

The amount of free water present in any other sample of damp sand of which the weight in air is known, and similarly weighed submerged, is the weight of the damp sample in air minus the constant previously determined; because the free water present in sand ceases to have any effect when the sample is weighed submerged.

Part of our procedure is to weigh exactly 2000 gr. of sand into a pan. The amount remaining after sufficient has been used, when weighed submerged, to balance the 1000 gr. weight, gives us the weight in air of the part used. The accuracy of the determination may be checked approximately by drying any sample. The dry weight should be the same as that of first sample.

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This method is as simple and rapid as any in use. It has the advantage that no special, expensive, or fragile apparatus is required, and that no charts or calculations are needed.

This method is a little simpler in application than the one outlined by Mr. Smith. It is accurate within the limits necessary, as it takes absorption into account; we find that absorption varies from ¾ to 2½ per cent in the sands used in our section. An error of 1 per cent in the value used as the absorption of the aggregates may cause a change in the cement content of a sack or more per cubic yard. The wetter and richer the mix, the greater will be the change.

Time is saved and errors avoided by using proportion instead of percentages to determine total free moisture in the aggregate. The free moisture in the batch is in direct proportion to that in the small sample. If weight proportioning is used, the weight of the batch is known; if volume proportioning is used, a close approximation can be made by weighing the amount of sand (or coarse aggregate) required to fill the 10-qt. bucket, which has a volume of ⅓ cu. ft. Care should be used to compact sand in the bucket as nearly as possible the same as by method of batching used.

In regard to drying sand to determine moisture, referred to in other papers, difficulty usually may be traced to unsuitable equipment. If the sample of damp sand, say 1000 gr., is spread out evenly in a good sized pan of the type known as a dripping pan, about 16 x 18 in., and dried over a suitable heater, it can be done in 3 or 4 min. without burning. Though we have used for this purpose heaters of many types, a favorite with us is a small gasoline heater known to auto tourists as a camp stove.

The aim in applying any of these methods should be to use a technique that is simple and accurate rather than one that is slow and tedious. I remember one time when I was with the Portland Cement Association I had occasion to help an engineer make moisture tests and sieve analyses of sand. Because the scales he had were not suitable, it was necessary for him to use a 5-lb. sample to get sufficient accuracy in either the moisture test or the sieve analysis. You can see how long it took him to dry out the 5-lb. sample and to put it through the sieves; he had a two hours' job. By using smaller samples, especially for the sieve analysis, splitting to two or three hundred grams and using a sensitive balance, both tests can be made in ten minutes.

Water Tables and Curves for Use in Designing and Estimating Concrete Mixtures

By Herbert J. Gilkey*

Introduction

Regardless of the particular theory or technique employed in the design of portland cement mixtures, the importance of the cement and the water upon the properties of the resultant product are such as to make their accurate measure an essential part of any design.

The language of water or of water and cement is a varied one. Each designer thinks in terms of units of a certain kind and he finds it difficult to coordinate his thought or compare his findings with the results of others who did not use the same code. The leading current units are as follows:

(1) Water-cement ratio by loose or bulk volume. (Abrams)
(2) Gallons of water per bag of cement.
(3) Pounds of water per bag of cement.
(4) Water-cement ratio by absolute volume.
(5) Ratio of water to cement by weight.

The purpose of the present paper is to:

(1) Supply convenient tables and figures covering each of the above units.

(2) Include the conversion element so that thought may not be restricted to one unit of measure. This makes the field and laboratory results more readily comparable one with another.

(3) Furnish for all units the corresponding estimated strengths according to the two well-known Abrams equations.

(4) Furnish strength ratios based upon the Abrams curves for added refinement in strength predictions according to the method outlined by the writer in "A Method for Predicting Concrete Strengths with Increased Precision."**

Four diagrams covering the same general features as the tables are included. Two of these are very similar to the Abrams curves, while the other two depart from them in form. All contain added features intended to assist in the more ready and better visualization that leads to improved perspective and exercise of better judgment.

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** *Proceedings*, American Concrete Institute, Vol. 24, 1928.

With both the tables and the diagrams, it has been the desire to meet the needs of the man in the specialized field as well as to cover the field as a whole. Therefore the tables are first given as one long all comprehensive table and then as separate tables suited for the usual more specialized needs. In like manner each type of diagram is furnished for the entire workable range of portland cement mixtures and also for the range of usual concretes and mortars.

NOTES—APPLICABLE TO ALL TABLES

Tables are computed from the following assumed constants or unit values: Heavy figures are basic assumptions. Light figures are computed from assumed values.

1 cu. ft. H_2O = 62.4 lb. = 7.48 gal.
1 cu. ft. Cem. (loose) = 94.0 lb. = 1 bag = 0.4782 cu. ft. (solid)
1 cu. ft. Cem. (solid) = 196.6 lb. = 2.091 bags = 2.091 cu. ft. (loose)
1 gal. H_2O = 8.342 lb. **Sp. Gr. Cement = 3.15**

For most persons to whom these data are of interest no explanations are necessary. For the sake of completeness it nevertheless seems to be worth while to include some explanatory matter and illustrative computations. The tables and figures will be treated in order.

EXPLANATIONS OF TABLES AND FIGURES

Table I—Conversion Factors for Different Water-Cement Units—These are the constants from which the other tables are computed and which can be used for conversion purposes. Attention is called to the assumed values for specific gravity of cement, unit weight of water, bulk or loose weight of cement per cubic foot or bag, and gallons of water per cubic foot. All of these are convenient or average approximations upon which all subsequent computations are based. In most cases the computations are carried one place beyond the correctness of the approximate values. The most frequent use of the tables will involve not more than two or three figures. The fourth is given to admit of close checking in refined calculations and there is no implication that the fourth figure is exactly right when it is derived from assumed values good to three places only.

Columns 5 and 6 represent water-cement ratios by weight and as such are applicable to metric as well as English measures. For example, the values can be considered as cubic centimeters of water per gram of cement or as liters per kilogram. Column 5 gives the ratio and column 6 gives the percentage or one hundred times the ratio.

Column 8 (W/C by absolute volume) is labelled Talbot simply to distinguish it from Abrams W/C which is by loose volume. This unit does not belong particularly to Talbot or any one else, but Talbot and Richart* make use of absolute volumes of all the constituents, including

* Bulletin 137, University of Illinois Engineering Experiment Station.

the water, and these units conform to their method of approaching the problem.

Table II—Equivalent Water-Cement Ratios, Estimated Strengths and Strength Ratios—This is a complete table which contains essentially everything that appears in Tables III to X inclusive. Slight approximations have been introduced by combining some of the values that were nearly identical. Even units in each column are emphasized by black-faced type. Detailed consideration of the several columns will be given in connection with the specialized partial tables that follow. These contain matter pertinent only to a particular unit and will be found more satisfactory for specialized use. All tables except X cover the entire workable range for portland cement mixtures.

Table III—Estimated Strengths and Strength Ratios for Rigid Control Using Formula $S = \frac{14{,}000}{7^x}$ (*Abrams*)—For those interested in finding the water-cement ratio (in any units) corresponding to any particular strength according to the Abrams estimate for rigidly controlled concrete, this table will be the most convenient. In like manner strength ratios based upon W/C = 1.00* are given in columns 3 and 4. Note that if it be desired to compare strengths on the basis of any other water-cement ratio taken as unity, the new set of ratios may be very easily computed from columns 1 or 2, or 3 or 4. Tables III and IV will often be used as a starting point in designing a concrete mixture of specified strength.

Table IV—Estimated Strengths and Strength Ratios for Average Job Using Formula $S = \frac{14{,}000}{9^x}$ (*Abrams*)—Identical with Table III except for equation used. See explanation for Table III.

Table V—Water-Cement Ratio by Loose Volume (*Abrams*)—This is the well-known water-cement ratio and is the unit most often implied when water-cement ratio is mentioned without other qualification.** The term loose volume arbitrarily means: cement assumed to weigh 94.0 lb. per cu. ft. and water 62.4 lb. per cu. ft. This is an average value for cement as sacked and is the net weight, presumably, of the cement in a sack which is assumed to contain one cubic foot. Bulk cement contains many weights per cubic foot, depending upon the method used for filling the measure. Poured in loosely it may weigh as little as 77 or 78 lb., while it is possible to crowd 100 lb. of bulk cement into a cubic foot container by tapping and jigging. It was such difficulties as these that led to the standardized approximation of 94 lb. per cu. ft. The sacking is performed by weighing.

The W/C units of this table are used about equally in the laboratory

* See "A Method for Predicting Concrete Strengths With Increased Precision." *Proceedings*, American Concrete Institute, Vol. 24, 1928.

** The latest A.C.I. Building Code (1928) and Concrete Reinforcing Steel Institute Code specify that water-cement ratio shall be expressed in U. S. gallons per bag of cement. The trend is definitely away from the loose volume unit of water-cement ratio and toward the gallons per bag unit.

and on the job, although the tendency is toward the use of gallons per bag for job concrete. They are not used within the normal consistency range (Table X).

Table VI—Gallons per Bag of Cement—A measure of water-cement ratio rarely used in the laboratory but very often used on the job and rapidly becoming standard. In fact, most water-cement ratio curves for job use show both the W/C by loose volume and the gallons per bag.* Note that one cubic foot of water is assumed to contain 7.48 gal. (7.5 in round numbers).

Table VII—Pounds of Water per Bag of Cement—A unit sometimes used on the job when the concrete is proportioned by weighing. Not a common unit.

Table VIII—Water-Cement Ratio by Absolute Volume—Not widely used but a fundamental unit likely to grow in importance as absolute volumes (i.e., the volumes as indicated by liquid displacement) become more generally familiar to designers of concrete mixtures. For fairly wet mixtures, the W/C by absolute volume closely approximates the voids-cement ratio, since in such mixtures the voids are largely water. The total voids will always slightly exceed the absolute water-cement ratio. For dry or stiff mixtures the water voids may not exceed 65 or 70 per cent of the total voids in the concrete or the mortar.** These tables are based upon an assumed specific gravity of 3.15 for cement, which gives a solid weight of 196.6 lb. per cu. ft., indicating that one cubic foot loose, as sacked (94.0 lb.), contains only 0.478 cu. ft. of cement absolute volume, i.e., it is more than half air (52.2 per cent). The specific gravity of portland cement may vary between 3.10 and 3.20, giving a possible error of 1.6 per cent due to the mean value assumed.

Table IX—Water-Cement Ratio by Weight and as Percentage by Weight—Note that this may be pounds of water per pound of cement; cubic centimeters or mililiters of water per gram of cement; liters per kilogram, etc.

This is the measure used in specifying the water to be used in connection with the standard tests for cement as regards both the neat cement and the 1:3 mortars. It is a common laboratory unit, but rarely if ever used elsewhere. The notable series of tests performed under the auspices of Committee C-1, American Society for Testing Materials, on 32 brands of cement by 52 laboratories specified the proportion of water to cement by weight. The fluid neat cement mixtures used contained 42 per cent of water by weight. This percentage is emphasized by black-faced type is Tables II, IX and X.

Table X—Normal Consistency Range—A detailed table covering the normal consistency range for both neat cements and the 1:3 standard sand mortars. Columns 10 and 13 and 11 and 13 are cross referenced to show readily the percentage of water required for the 1:3 standard

* The latest A.C.I. Building Code (1928) specifies that water-cement ratio be expressed as U. S. gallons per bag of cement.

** See Bulletin 137, University of Illinois Engineering Experiment Station.

sand mortar, corresponding to the normal consistency of the neat cement, and vice versa. The 42 per cent ratio is emphasized across the table as mentioned in connection with the explanation of Table IX.

Figure 1—Amplified Water-Cement Ratio and Strength Chart Covering the Entire Workable Range of Portland Cement Mixtures—This is but the usual Abrams curve with a few embellishments. It embodies in graphical form about the same information as the tables and may be used in the same way. Besides these general features it is contoured to show at a glance the effect of increasing or decreasing the water-cement ratio at intervals of 10 per cent. This feature is designed to give added perspective in mental evaluation or visualization. It must be realized that for given proportions of specified materials, the range of possible variation within the zone of suitable workability is likely to be restricted to an optional 10 or 20 per cent either way, i.e., a mixture having a slump of say 3 in. is likely to be entirely unworkable with a 20 per cent lower water-cement ratio, and unduly wet with a 20 per cent greater one. A considerable range may be attained by variation of aggregate-cement ratio. It should be further kept in mind that the strengths given are but approximate, since the same water-cement ratios for different cements, aggregates, temperatures of curing and conditions of placing may produce strengths differing by 50 to 100 per cent from those shown here. For such stronger or weaker mixtures variations in the water-cement ratios will produce effects of comparable magnitude. The strength ratios are useful in diminishing the spreads between predicted and actual strengths when once the strength of a mixture is known at one or more water-cement ratios. This phase has been fully covered elsewhere in this paper and in "A Method for Predicting Concrete Strengths with Increased Precision" *Proceedings*, American Concrete Institute, 1928. The different water-cement ratio scales given below the figure will facilitate thought in more than one set of current units.

Figure 2—Amplified Water-Cement Ratio and Strength Chart Covering the Workable Range of Portland Cement Concretes—This is similar to Fig. 1, but should be of more use to the man primarily interested in the "concrete on the job." The range is restricted to that of usual concrete mixtures and the extra curves show at a glance the effects of increases or decreases in cement and water expressed in the kind of units that the contractor is accustomed to using. Otherwise the figure is similar to Fig. 1 and all that has been said of Fig. 1, applies to Fig. 2.

Figure 3—Straight Line Water-Cement Ratio and Strength Chart Covering the Entire Workable Range of Portland Cement Mixtures—In general use and scope the information of Fig. 3 is identical with that of the Tables II–X inclusive and Fig. 1. The material is shown in a slightly different form that for some purposes and persons may prove to be preferable. The estimated strengths and strength ratios for the two Abrams equations appear on the diagonal scale. The chart may be entered from top, bottom, or either side via the water-cement units desired. Strength estimates and strength ratios are read directly from

the diagonal line, while conversion to other units is attained by running from the diagonal line to the desired scale.

Figure 4—Straight Line Water and Strength Chart Covering the Workable Range of Portland Cement Concretes—Similar to Fig. 3, but restricted to the range of usual concrete mixtures. See explanation of Fig. 3 for further details.

ILLUSTRATIVE USE OF UNITS AND RATIOS

Standard laboratory tests were made of concrete from given cement and aggregates. Strength at 28 days (standard curing) was 3000 lb. per sq. in. Water is reported as 55 per cent of cement by weight.

(*a*) What does this water content mean in terms of gallons per bag?

(*b*) What strength should one expect from this concrete at 7 gal. per bag if rigidly controlled?

(*c*) What job strength should be expected for average conditions?

Solution:

(*a*) Using Table II or Table IX, Col. 10 and 6, 55 per cent by weight equals 6.2 gal. per bag. In like manner any of the figures could be used or the conversion could be made by the factors of Table I. Estimated strength for this water ratio is 2790 (Col. 1, rigidly controlled) or 2270 (Col. 2, average job).

(*b*) Strength ratio (Col 3) is 1.40, while for 7 gal. per bag (Col. 6, Tables II or VI) the ratio is 1.13 (Col. 3). Thus the probable rigidly controlled strength will be $1.13/1.40 \times 3000 = 2420$ lb. per sq. in. instead of 2270 lb. per sq. in., the estimated strength for 7 gal. per bag.

(*c*) At this water-cement ratio it will be noted that the average job concrete is running about 475 lb. per sq. in. below the rigidly controlled (Cols. 1 and 2) and it is therefore close enough to assume that the job concrete from these materials would have a 28-day standard strength of 1950 lb. per sq. in. instead of the estimated 1790.

Remarks:

Note that these ratios assume perfect parallelism with the Abrams curves. Insofar as the water-cement ratio curve for the concrete in question fails to parallel the Abrams relation there will be error introduced in this type of prediction. The error will be very slight if the new water-cement ratio for which the strength is being predicted does not differ greatly from that for which test results are available. In any case the use of the ratios will furnish a much closer result than will be obtained from strict adherence to the estimated strengths of Cols. 1 and 2.

Some accomplish a similar refinement through a slightly different procedure. Lord* assumes that the water-cement relation is not expressed by a curve parallel to either Abrams curve but by one of identical form, i.e., $S = \frac{14{,}000}{C^x}$, where C may have any value ranging in general

* "Notes on Wacker Drive," *Proceedings*, American Concrete Institute, Vol. 23, 1927, p. 40.

from 9 to 4. The lower the value of C, the higher the curve is lifted, but the lifted curve will be nearly but not quite parallel to other curves having different C values. Even if this procedure be more nearly correct than the assumption of parallelism, the difference between the two will be very slight when the water ratio of the trial mixture does not depart widely from that being predicted or designed.

Some will find many features of value. Others will prefer to reject or ignore certain features and make use of others. If these tables and charts assist in clarifying a varied nomenclature or in abbreviating the work of designing or estimating mixtures, their purpose has been served. The writer feels that this is but one of a variety of needed steps to promote a better understanding and improved control leading to safer, better and more economical concrete construction.

Sartwell Egerton, civil engineering student, has aided in the preparation of the tables and figures. J. M. Buirgy, also a student in civil engineering, assisted in the preparation. Professor C. L. Eckel, head of the civil engineering department, has always maintained an attitude of sympathetic encouragement toward such projects. Of course, the entire basis for strength estimates offered herein is the work of Abrams and the laboratory of the Portland Cement Association. It is felt that this project is strictly in line with the admirable type of educational promotion that they have so effectively conducted.

TABLE I.—CONVERSION FACTORS FOR DIFFERENT WATER-CEMENT UNITS

	W/C (loose) (Abrams)	Gal. per Bag	Lb. per Bag	Lb. per Lb.	Per Cent Weight Cement	Per Cent Weight Cement plus Sand 1:3	W/C (absolute) (Talbot)
1	2	3	4	5	6	7	8
W/C (loose) (Abrams) to	1	7.480	62.40	0.6638	66.38	16.60	2.091
Gal. per Bag to	0.1337	1	8.342	0.08874	8.874	2.219	0.2795
Lb. per Bag to	0.01603	0.1199	1	0.01064	1.064	0.2660	0.03351
Lb. per Lb. to	1.506	11.27	94.00	1	100.0	25.00	3.150
Per Cent Weight Cement to	0.01506	0.1127	0.9400	0.0100	1	0.2500	0.0315
Per Cent Weight Cement plus Sand 1:3 to	0.06024	0.4506	3.759	0.0400	4.000	1	0.1260
W/C (absolute) (Talbot) to	0.4782	3.577	29.84	0.3175	31.75	7.936	1

TABLE II.—EQUIVALENT WATER-CEMENT RATIOS, ESTIMATED STRENGTHS AND STRENGTH RATIOS

Strength, 28-Day Standard, Approximate (Abrams)		Ratio of Strength to that for W/C = 1		Ratio of Water to Cement Expressed in Different Recognized Units							General	
Rigid Control $S = \frac{14,000}{7x}$	Average Job $S = \frac{14,000}{9x}$	$S = \frac{14,000}{7x}$	$S = \frac{14,000}{9x}$	W/C by Loose Volume (Abrams)	Gallons per Bag of Cement	Lb. per Bag of Cement	W/C by Absolute Volume (Talbot)	W/C by Weight (lb./lb.; l./kg.; c.c./g.; etc.)	Per Cent Cement by Weight (100 × Col. 9)	Per Cent Weight Cement plus Sand in 1:3 Mixtures	Description and Remarks	Cross Reference to Per Cent H_2O for Standard Mixture (1:3 above)(neat below)
1	2	3	4	5	6	7	8	9	10	11	12	13
Strength		Strength Ratios										
Rigid	Aver. Job	Rigid	Aver. Job	W/C (Abrams)	Gal. per Bag	Lb. per Bag	W/C Absolute (Talbot)	W/C by Wt.	Per Cent by Wt.	1:3 Per Cent Total Wt.		
9020	8523	4.560	5.481	.2259	1.690	14.10	.4724	.1500	15.00	....	Other Mixtures too Dry to Work.	9.0
9000	8500	4.500	5.468	.2269	1.697	14.16	.4743	.1506	15.06	....		....
8759	8244	4.380	5.302	.2410	1.803	15.00	.5029	.1600	16.00	....		9.2
8607	8083	4.304	5.198	.2500	1.870	15.60	.5228	.1660	16.60	....		....
8529	8000	4.265	5.145	.2547	1.905	15.89	.5326	.1691	16.91	....		....
8500	7975	4.250	5.129	.2561	1.916	15.98	.5355	.1700	17.00	....		9.3
8320	7780	4.160	5.000	.2674	2.000	16.69	.5591	.1775	17.75	....		....
8262	7719	4.131	4.964	.2710	2.027	16.91	.5667	.1800	18.00	....		9.5
8056	7500	4.028	4.823	.2840	2.124	17.72	.5938	.1885	18.85	....		....
8020	7464	4.010	4.800	.2863	2.142	17.90	.6000	.1900	19.00	....		9.7
8000	7439	4.000	4.784	.2878	2.153	17.96	.6018	.1910	19.10	....		....
7827	7242	3.914	4.657	.3000	2.244	18.72	.6273	.1991	19.91	....		....
7791	7223	3.896	4.645	.3012	2.253	18.79	.6298	.2000	20.00	....		9.8
7725	7153	3.863	4.600	.3056	2.286	19.07	.6390	.2029	20.29	....		....
7580	7000	3.790	4.502	.3153	2.358	19.67	.6593	.2093	20.93	....		....
7565	6987	3.783	4.493	.3163	2.366	19.74	.6614	.2100	21.00	....		10.0
7502	6922	3.751	4.451	.3206	2.398	20.00	.6704	.2128	21.28	....		....
7500	6916	3.750	4.448	.3210	2.401	20.03	.6712	.2131	21.31	....		....
7425	6842	3.713	4.400	.3259	2.438	20.34	.6815	.2163	21.63	....		....
7348	6761	3.674	4.347	.3313	2.478	20.67	.6927	.2200	22.00	....		10.2
7305	6716	3.653	4.319	.3343	2.500	20.86	.6990	.2219	22.19	....		....
7135	6540	3.568	4.200	.3470	2.596	21.65	.7256	.2300	23.00	....	Normal Consistency Range for Neat Cements.	10.3
7100	6500	3.550	4.180	.3490	2.611	21.78	.7298	.2317	23.17	....		....
7085	6489	3.543	4.173	.3500	2.618	21.84	.7319	.2323	23.23	....		....
7000	6404	3.500	4.118	.3560	2.663	22.21	.7444	.2363	23.63	....		....
6930	6328	3.465	4.069	.3614	2.703	22.55	.7557	.2400	24.00	....		10.5
6825	6220	3.413	4.000	.3692	2.762	23.04	.7720	.2451	24.51	....		....
6729	6122	3.365	3.937	.3765	2.816	23.49	.7873	.2500	25.00	....		10.7
6650	6040	3.325	3.884	.3826	2.862	23.87	.8000	.2540	25.40	....		....
6611	6000	3.306	3.859	.3856	2.884	24.06	.8063	.2560	25.60	....		....
6534	5922	3.267	3.800	.3926	2.937	24.50	.8209	.2600	26.00	....		10.8
6500	5891	3.250	3.788	.3940	2.947	24.59	.8239	.2615	26.15	....		....
6420	5800	3.207	3.729	.4000	3.000	25.00	.8387	.2663	26.63	....		....
6346	5730	3.173	3.685	.4066	3.041	25.37	.8502	.2700	27.00	....		11.0
6219	5600	3.110	3.600	.4172	3.121	26.03	.8724	.2769	27.69	....		....
6162	5543	3.081	3.565	.4217	3.154	26.31	.8818	.2800	28.00	....		11.2
6123	5500	3.062	3.537	.4250	3.179	26.52	.8887	.2821	28.21	....		....
6021	5400	3.011	3.473	.4336	3.243	27.06	.9067	.2878	28.78	....		....
6000	5378	3.000	3.459	.4354	3.257	27.17	.9104	.2890	28.90	....		....
5985	5363	2.993	3.449	.4367	3.267	27.25	.9131	.2900	29.00	....		11.3
5910	5287	2.955	3.400	.4432	3.315	27.66	.9267	.2942	29.42	....		....
5820	5200	2.912	3.350	.4500	3.366	28.08	.9410	.3000	30.00	....		11.5
5800	5177	2.900	3.329	.4528	3.387	28.25	.9468	.3006	30.06	7.516	....	....
5644	5019	2.822	3.228	.4669	3.492	29.13	.9763	.3100	31.00	7.751	....	....
5625	5000	2.813	3.215	.4680	3.500	29.20	.9786	.3107	31.07	7.769	....	....
5600	4975	2.800	3.200	.4708	3.522	29.38	.9844	.3125	31.25	7.815	....	....
5521	4896	2.762	3.149	.4782	3.577	29.84	1.000	.3175	31.75	7.936	....	....
5500	4876	2.750	3.136	.4800	3.590	30.00	1.006	.3192	31.92	7.983	....	....

H_2O to be Used for 1:3 Standard Sand Mixture at this Normal Consistency (Expressed as Percentage of Cement + Standard Sand by Weight)

TABLE II.—EQUIVALENT WATER-CEMENT RATIOS, ESTIMATED STRENGTHS AND STRENGTH RATIOS.—(Continued)

Strength, 28-Day Standard, Approximate (Abrams)		Ratio of Strength to that for W/C = 1		Ratio of Water to Cement Expressed in Different Recognized Units							General	
Rigid Control $S = \frac{14,000}{7^x}$	Average Job $S = \frac{14,000}{9^x}$	$S = \frac{14,000}{7^x}$	$S = \frac{14,000}{9^x}$	W/C by Loose Volume (Abrams)	Gallons per Bag of Cement	Lb. per Bag of Cement	W/C by Absolute Volume (Talbot)	W/C by Weight (lb./lb.; l./kg.; c.c./g.; etc.)	Per Cent Cement by Weight (100 × Col. 9)	Per Cent Weight Cement plus Sand in 1:3 Mixtures	Description and Remarks	Cross Reference to Per Cent H_2O for Standard Mixture (1:3 above)(neat below)
1	2	3	4	5	6	7	8	9	10	11	12	13
Strength		Strength Ratios										
Rigid	Aver. Job	Rigid	Aver. Job	W/C (Abrams)	Gal. per Bag	Lb. per Bag	W/C Absolute (Talbot)	W/C by Wt.	Per Cent by Wt.	1:3 Per Cent Total Wt.		
5481	4856	2.741	3.123	.4819	3.605	30.07	1.008	.3200	32.00	8.000	....	....
5425	4800	2.713	3.087	.4872	3.644	30.40	1.019	.3234	32.34	8.088	....	....
5400	4775	2.700	3.071	.4896	3.662	30.55	1.024	.3250	32.50	8.127	....	....
5323	4698	2.662	3.021	.4970	3.718	31.01	1.039	.3300	33.00	8.250	....	....
5289	4665	2.645	3.000	.5000	3.740	31.20	1.046	.3320	33.20	8.300	....	....
5225	4600	2.613	2.958	.5065	3.789	31.61	1.059	.3362	33.62	8.408	....	....
5200	4575	2.600	2.942	.5090	3.807	31.76	1.064	.3379	33.79	8.449	....	....
5169	4545	2.585	2.923	.5120	3.830	31.95	1.071	.3400	34.00	8.499	....	....
5129	4500	2.565	2.894	.5160	3.860	32.20	1.079	.3425	34.25	8.566	....	....
5020	4400	2.510	2.830	.5268	3.940	32.87	1.102	.3500	35.00	8.750	....	....
5000	4378	2.500	2.815	.5291	3.958	33.02	1.106	.3512	35.12	8.783	....	....
4977	4345	2.489	2.800	.5315	3.976	33.17	1.111	.3528	35.28	8.823	....	....
4945	4323	2.473	2.780	.5348	4.000	33.37	1.118	.3550	35.50	8.878	....	....
4874	4253	2.437	2.735	.5422	4.056	33.83	1.134	.3600	36.00	9.000	Normal Consistency Range for 1:3 Standard Sand Mortars	15.0
4846	4226	2.423	2.718	.5452	4.078	34.02	1.140	.3620	36.20	9.050		....
4820	4200	2.410	2.700	.5480	4.100	34.20	1.146	.3640	36.40	9.100		....
4812	4181	2.406	2.689	.5500	4.114	34.32	1.150	.3651	36.51	9.130		....
4700	4180	2.400	2.688	.5501	4.115	34.33	1.150	.3660	36.60	9.150		....
4762	4143	2.381	2.664	.5542	4.145	34.58	1.159	.3680	36.80	9.200		16.0
4734	4116	2.367	2.647	.5572	4.168	34.77	1.165	.3700	37.00	9.250		....
4800	4080	2.349	2.630	.5602	4.190	35.00	1.173	.3720	37.20	9.300		17.0
4679	4062	2.340	2.612	.5632	4.213	35.14	1.178	.3740	37.40	9.349		....
4660	4043	2.331	2.600	.5652	4.228	35.27	1.182	.3760	37.60	9.400		....
4625	4000	2.308	2.572	.5702	4.265	35.58	1.190	.3780	37.80	9.449		....
4600	3984	2.300	2.560	.5720	4.279	35.69	1.196	.3800	38.00	9.500		18.0
4584	3968	2.292	2.552	.5738	4.292	35.81	1.200	.3809	38.09	9.525		....
4570	3955	2.285	2.543	.5753	4.303	35.90	1.203	.3820	38.20	9.550		....
4544	3929	2.272	2.527	.5783	4.326	36.09	1.209	.3840	38.40	9.600		....
4517	3903	2.258	2.510	.5813	4.348	36.27	1.215	.3860	38.60	9.650		....
4500	3889	2.250	2.501	.5830	4.361	36.38	1.219	.3880	38.80	9.700		19.0
4465	3852	2.233	2.477	.5873	4.393	36.65	1.228	.3900	39.00	9.749		....
4439	3827	2.220	2.461	.5903	4.415	36.83	1.234	.3920	39.20	9.800		20.0
4411	3800	2.206	2.444	.5935	4.439	37.03	1.241	.3940	39.40	9.852		....
4400	3789	2.200	2.437	.5948	4.449	37.12	1.244	.3948	39.48	9.873		....
4387	3777	2.194	2.429	.5963	4.460	37.21	1.247	.3960	39.60	9.900		....
4356	3746	2.178	2.409	.6000	4.488	37.40	1.253	.3980	39.80	9.960	Stiff or Rich Concretes—Usual Mortars	....
4336	3727	2.168	2.400	.6024	4.500	37.59	1.260	.4000	40.00	10.00		21.0
4310	3702	2.155	2.381	.6054	4.528	37.78	1.266	.4020	40.20	10.05		....
4285	3678	2.143	2.365	.6084	4.550	37.96	1.273	.4040	40.40	10.10		....
4260	3654	2.130	2.350	.6114	4.573	38.15	1.278	.4060	40.60	10.15		....
4235	3629	2.118	2.334	.6144	4.596	38.34	1.285	.4080	40.80	10.20		22.0
4200	3600	2.100	2.312	.6187	4.628	38.61	1.294	.4100	41.00	10.25		....
4186	3581	2.093	2.303	.6205	4.641	38.72	1.297	.4120	41.20	10.30		23.0
4161	3558	2.081	2.288	.6235	4.664	38.91	1.304	.4140	41.40	10.35		....
4137	3534	2.068	2.273	.6265	4.686	39.09	1.310	.4160	41.60	10.40		....
4103	3500	2.052	2.251	.6307	4.718	39.36	1.319	.4180	41.80	10.45		....
4089	3488	2.045	2.243	.6325	4.731	39.47	1.323	.4200	42.00	10.50		24.0
4065	3465	2.033	2.228	.6355	4.754	39.66	1.329	.4220	42.20	10.55		....
4041	3442	2.021	2.214	.6385	4.776	39.84	1.335	.4240	42.40	10.60		....
4020	3420	2.010	2.201	.6412	4.796	40.00	1.341	.4260	42.60	10.65		....
4000	3400	2.000	2.188	.6438	4.816	40.17	1.346	.4280	42.80	10.70		25.0
3970	3374	1.985	2.170	.6476	4.843	40.41	1.354	.4300	43.00	10.75		....

Normal Consistency of Neat Cement Corresponding to this Water Requirement for 1:3 Standard Sand Mortar

TABLE II.—EQUIVALENT WATER-CEMENT RATIOS, ESTIMATED STRENGTHS AND STRENGTH RATIOS.—(Continued)

Strength, 28-Day Standard, Approximate (Abrams)		Ratio of Strength to that for W/C = 1		Ratio of Water to Cement Expressed in Different Recognized Units							General	
Rigid Control $S = \frac{14,000}{7x}$	Average Job $S = \frac{14,000}{9x}$	$S = \frac{14,000}{7x}$	$S = \frac{14,000}{9x}$	W/C by Loose Volume (Abrams)	Gallons per Bag of Cement	Lb. per Bag of Cement	W/C by Absolute Volume (Talbot)	W/C by Weight (lb./lb.; l./kg.; c.c./g.; etc.)	Per Cent Cement by Weight ($100 \times$ Col. 9)	Per Cent Weight Cement plus Sand in 1:3 Mixtures	Description and Remarks	Cross Reference to Per Cent H_2O for Standard Mixture (1:3 above) (neat below)
1	2	3	4	5	6	7	8	9	10	11	12	13
Strength		Strength Ratios										
Rigid	Aver. Job	Rigid	Aver. Job	W/C (Abrams)	Gal. per Bag	Lb. per Bag	W/C Absolute (Talbot)	W/C by Wt.	Per Cent by Wt.	1:3 Per Cent Total Wt.		
3952	3356	1.976	2.158	.6500	4.862	40.60	1.360	.4320	43.20	10.80	Normal Consistency Range for 1:3 Standard Sand Motors (3952–3637); Stiff or Pich Concretes—Usual Mortars (3952–2580)	26.0
3924	3330	1.962	2.141	.6536	4.889	40.78	1.367	.4340	43.40	10.85		....
3902	3308	1.951	2.127	.6566	4.911	40.97	1.373	.4360	43.60	10.90		....
3879	3286	1.940	2.113	.6596	4.934	41.16	1.379	.4380	43.80	10.95		....
3844	3250	1.922	2.100	.6626	4.956	41.35	1.386	.4400	44.00	11.00		27.0
3800	3211	1.900	2.068	.6695	5.000	41.71	1.400	.4440	44.40	11.10		....
3789	3200	1.895	2.058	.6717	5.024	41.91	1.405	.4460	44.60	11.15		....
3767	3180	1.884	2.045	.6746	5.046	42.10	1.411	.4480	44.80	11.20		28.0
3745	3158	1.873	2.031	.6777	5.069	42.29	1.417	.4500	45.00	11.25		....
3723	3138	1.862	2.018	.6807	5.092	42.48	1.423	.4520	45.20	11.30		29.0
3700	3100	1.840	2.000	.6847	5.122	42.73	1.432	.4560	45.60	11.40		....
3658	3076	1.829	1.978	.6897	5.159	43.04	1.442	.4580	45.80	11.45		....
3637	3056	1.818	1.965	.6927	5.181	43.22	1.448	.4600	46.00	11.50		30.0
3600	3021	1.800	1.943	.6979	5.220	43.55	1.459	.4633	46.33	11.59		....
3578	3000	1.789	1.929	.7000	5.236	43.75	1.466	.4654	46.54	11.64		....
3500	2929	1.750	1.884	.7120	5.326	44.43	1.489	.4726	47.26	11.82		....
3439	2869	1.720	1.845	.7214	5.396	45.00	1.508	.4789	47.89	11.98		....
3400	2832	1.700	1.821	.7273	5.440	45.38	1.521	.4828	48.28	12.07		....
3365	2800	1.683	1.800	.7325	5.480	45.71	1.532	.4863	48.63	12.16		....
3347	2782	1.674	1.789	.7354	5.500	45.89	1.538	.4882	48.82	12.21		....
3311	2750	1.656	1.768	.7410	5.543	46.24	1.549	.4919	49.19	12.30		....
3250	2694	1.625	1.732	.7500	5.610	46.80	1.568	.4979	49.79	12.45		....
3234	2677	1.617	1.722	.7530	5.632	46.99	1.575	.5000	50.00	12.50		....
3200	2644	1.600	1.700	.7585	5.674	47.33	1.586	.5035	50.35	12.59		....
3152	2600	1.576	1.672	.7662	5.731	47.81	1.600	.5079	50.79	12.70		....
3045	2500	1.523	1.608	.7840	5.864	48.92	1.639	.5204	52.04	13.01		....
3031	2488	1.516	1.600	.7863	5.882	49.07	1.644	.5219	52.19	13.05		....
3000	2459	1.500	1.581	.7916	5.921	49.40	1.655	.5255	52.55	13.14		....
2940	2400	1.470	1.545	.8000	6.000	50.00	1.676	.5320	53.20	13.30	Usual Concretes (2940–1989)	....
2800	2275	1.400	1.463	.8270	6.186	51.60	1.729	.5500	55.00	13.75		....
2776	2250	1.388	1.447	.8315	6.220	51.89	1.739	.5519	55.19	13.80		....
2750	2230	1.375	1.434	.8360	6.253	52.17	1.748	.5549	55.49	13.88		....
2719	2200	1.360	1.415	.8422	6.300	52.55	1.761	.5591	55.91	13.98		....
2694	2177	1.347	1.400	.8470	6.336	52.85	1.771	.5622	56.22	14.06		....
2678	2163	1.339	1.391	.8500	6.358	53.04	1.777	.5642	56.42	14.11		....
2622	2112	1.311	1.358	.8608	6.439	53.71	1.800	.5714	57.14	14.29		....
2600	2092	1.300	1.345	.8652	6.472	53.99	1.809	.5743	57.43	14.36		....
2580	2074	1.290	1.334	.8691	6.500	54.23	1.817	.5769	57.69	14.43		....
2500	2000	1.250	1.286	.8850	6.620	55.00	1.844	.5853	58.53	14.64		....
2430	1938	1.215	1.246	.9000	6.732	56.16	1.882	.5974	59.74	14.94		....
2400	1911	1.200	1.229	.9063	6.779	56.55	1.895	.6000	60.00	15.00		....
2350	1866	1.175	1.200	.9172	6.861	57.23	1.918	.6088	60.88	15.23		....
2266	1792	1.133	1.152	.9359	7.000	58.40	1.957	.6213	62.13	15.54		....
2250	1783	1.125	1.147	.9380	7.016	58.53	1.961	.6226	62.26	15.57		....
2222	1750	1.111	1.125	.9460	7.067	59.03	1.978	.6280	62.80	15.70		....
2204	1732	1.100	1.114	.9500	7.106	59.28	1.986	.6306	63.06	15.77		....
2177	1712	1.089	1.101	.9564	7.154	59.70	2.000	.6349	63.49	15.88		....
2159	1692	1.080	1.088	.9618	7.194	60.00	2.011	.6384	63.84	15.97		....
2084	1630	1.042	1.048	.9789	7.322	61.08	2.047	.6500	65.00	16.25		....
2050	1600	1.025	1.029	.9872	7.384	61.60	2.064	.6553	65.53	16.39		....
2000	1555	1.000	1.000	1.000	7.480	62.40	2.091	.6638	66.38	16.60		....
1989	1546	.9945	.9942	1.003	7.500	62.57	2.097	.6657	66.57	16.65		....

Normal Consistency of Neat Cement Corresponding to this Water Requirement for 1:3 Standard Sand Mortar

TABLE II.—EQUIVALENT WATER-CEMENT RATIOS, ESTIMATED STRENGTHS AND STRENGTH RATIOS.—(Continued)

Strength, 28-Day Standard, Approximate (Abrams)		Ratio of Strength to that for W/C = 1		Ratio of Water to Cement Expressed in Different Recognized Units							General	
Rigid Control $S = \frac{14{,}000}{7x}$	Average Job $S = \frac{14{,}000}{9x}$	$S = \frac{14{,}000}{7x}$	$S = \frac{14{,}000}{9x}$	W/C by Loose Volume (Abrams)	Gallons per Bag of Cement	Lb. per Bag of Cement	W/C by Absolute Volume (Talbot)	W/C by Weight (lb./lb.; l./kg.; c.c./g.; etc.)	Per Cent Cement by Weight (100 × Col. 9)	Per Cent Weight Cement plus Sand in 1:3 Mixtures	Description and Remarks	Cross Reference to Per Cent H_2O for Standard Mixture (1:3 above)(neat below)
1	2	3	4	5	6	7	8	9	10	11	12	13
Strength		Strength Ratios										
Rigid	Aver. Job	Rigid	Aver. Job	W/C (Abrams)	Gal. per Bag	Lb. per Bag	W/C Absolute (Talbot)	W/C by Wt.	Per Cent by Wt.	1:3 Per Cent Total Wt.		
1935	**1500**	.9675	.9646	1.017	7.607	63.46	2.127	.6751	67.51	16.88	Usual Concretes Cont'd	....
1843	1418	.9215	.9119	1.042	7.794	**65.00**	2.179	.6917	69.17	17.30		....
1822	1400	.9110	.9003	**1.050**	7.854	65.40	2.191	.6970	69.70	17.40		....
1800	1383	.9000	.8894	1.054	7.884	65.77	**2.200**	**.7000**	**70.00**	17.50		....
1750	1340	**.8750**	.8617	1.068	**8.000**	66.77	2.233	.7089	70.89	17.73		....
1650	**1250**	.8200	.8000	**1.100**	8.228	68.64	2.300	.7302	73.02	18.26		....
1600	1200	.8000	.7768	1.115	8.340	**70.00**	2.331	.7401	74.01	18.51		....
1555	1169	.7775	.7518	1.130	8.452	70.51	2.363	**.7500**	**75.00**	18.76		....
1532	1151	.7660	.7402	1.137	**8.500**	70.95	2.377	.7547	75.47	18.87		....
1500	1126	**.7500**	.7241	**1.150**	8.580	71.57	**2.400**	.7620	76.20	19.06		....
1400	1041	.7000	.6695	1.183	8.849	73.82	2.474	.7853	78.53	19.64		....
1350	**1000**	.6750	.6405	**1.200**	**9.000**	**75.00**	2.520	**.8000**	**80.00**	20.00		....
1271	933	.6355	.6000	1.233	9.223	76.94	2.578	.8185	81.85	20.47		....
1250	914	**.6250**	.5878	1.242	9.290	77.50	**2.600**	.8251	82.51	20.63		....
1230	900	.6150	.5775	**1.250**	9.350	78.00	2.614	.8298	82.98	20.75		....
1200	873	.6000	.5614	1.263	9.447	78.81	2.641	.8384	83.84	20.97		....
1183	860	.5915	.5531	1.270	**9.500**	79.25	2.656	.8430	84.30	21.08		....
1160	841	.5800	.5408	1.280	9.574	**80.00**	2.681	**.8500**	**85.00**	21.25		....
1111	800	.5580	.5177	**1.300**	9.724	81.12	2.718	.8629	86.29	21.58		....
1050	**750**	.5250	.4823	1.337	**10.00**	83.43	**2.800**	.8875	88.75	22.19		....
1000	712	**.5000**	.4585	**1.350**	10.10	84.55	2.833	**.9000**	**90.00**	22.49		....
987	700	.4935	.4508	1.363	10.20	**85.00**	2.850	.9048	90.48	22.63		....
918	646	.4590	.4154	**1.400**	**10.50**	87.61	2.936	.9320	93.20	23.31		....
900	632	.4500	.4000	1.410	10.60	87.98	2.948	.9360	93.60	23.41		....
860	600	.4300	.3859	1.434	10.70	89.48	**3.000**	**.9500**	**95.00**	23.80	Very Wet or Lean Concretes	....
840	579	.4165	.3723	**1.450**	10.85	**90.00**	3.017	.9579	95.79	23.95		....
800	553	.4000	.3556	1.471	**11.00**	91.79	3.076	.9764	97.64	24.42		....
750	515	**.3750**	.3312	**1.500**	11.24	93.79	3.143	.9977	99.77	24.95		....
747	512	.3735	.3293	1.506	11.27	94.00	3.149	**1.000**	**100.0**	25.00		....
731	**500**	.3655	.3170	1.523	11.39	**95.00**	3.185	1.011	101.1	25.28		....
700	476	.3500	.3061	1.539	**11.50**	95.97	**3.200**	1.021	102.1	25.53		....
686	465	.3430	.2990	**1.550**	11.59	96.72	3.241	1.029	102.9	25.73		....
646	434	.3230	.2791	1.581	11.83	98.65	3.306	**1.050**	**105.0**	26.24		....
620	414	.3095	.2662	**1.600**	**12.00**	**100.0**	3.352	1.064	106.4	26.61		....
600	400	.3000	.2566	1.619	12.10	101.0	**3.400**	1.075	107.5	26.88		....
560	373	.2825	.2360	**1.650**	12.34	103.0	3.450	**1.100**	**110.0**	27.51		....
530	347	.2650	.2232	1.683	**12.50**	**105.0**	3.519	1.117	111.7	29.94		....
500	325	**.2500**	.2090	**1.700**	12.81	106.8	**3.600**	1.136	113.6	28.42		....
470	**300**	.2380	.2000	**1.750**	**13.00**	108.5	3.634	**1.150**	**115.0**	28.75		....
453	291	.2265	.1871	1.763	13.19	**110.0**	3.686	1.170	117.0	29.27		....
420	265	.2090	.1704	**1.800**	**13.50**	112.6	3.774	**1.200**	**120.0**	30.00		....
400	253	.2000	.1627	1.827	13.67	114.0	**3.800**	1.206	120.6	30.16		....
383	240	.1915	.1543	**1.850**	13.84	**115.0**	3.868	1.228	122.8	30.71		....
360	224	.1795	.1441	**1.900**	**14.00**	117.5	3.937	**1.250**	**125.0**	31.26		....
330	200	.1625	.1286	1.934	**14.50**	**120.0**	**4.000**	1.270	127.0	31.76		....
300	190	**.1500**	.1177	**1.950**	14.65	122.2	4.094	**1.300**	**130.0**	32.50		....
282	171	.1410	.1100	**2.000**	**15.00**	**125.0**	**4.200**	1.330	133.0	33.30		....
268	161	.1340	.1035	2.033	15.21	126.9	4.251	**1.350**	**135.0**	33.75		....
200	116	.1000	.0746	2.183	16.63	136.2	4.565	1.449	144.9	36.24		....

TABLE III.—ESTIMATED STRENGTHS AND STRENGTH RATIOS FOR RIGID CONTROL, USING $S = \frac{14,000}{7^x}$ (ABRAMS)

Strength, 28-Day Standard, Approximate (Abrams)		Ratio of Strength to that for W/C = 1		Ratio of Water to Cement Expressed in Different Recognized Units							General
Rigid Control $S = \frac{14,000}{7x}$	Average Job $S = \frac{14,000}{9x}$	$S = \frac{14,000}{7x}$	$S = \frac{14,000}{9x}$	W/C by Loose Volume (Abrams)	Gallons per Bag of Cement	Lb. per Bag of Cement	W/C by Absolute Volume (Talbot)	W/C by Weight (lb./lb.; l./kg.; c.c./g.; etc.)	Per Cent Cement by Weight (100 × Col. 9)	Per Cent Weight Cement plus Sand in 1:3 Mixtures	Description and Remarks
1	2	3	4	5	6	7	8	9	10	11	12
Strength		Strength Ratios									
Rigid	Aver. Job	Rigid	Aver. Job	W/C (Abrams)	Gal. per Bag	Lb. per Bag	W/C Absolute (Talbot)	W/C by Wt.	Per Cent by Wt.	1:3 Per Cent Total Wt.	
9000	8500	**4.500**	5.468	.2269	1.697	14.16	.4743	.1506	15.06	....	
8500	7975	**4.250**	5.129	.2561	1.916	15.98	.5355	.1700	17.00	....	
8000	7439	**4.000**	4.784	.2878	2.153	17.96	.6018	.1910	19.10	....	
7500	6916	**3.750**	4.448	.3210	2.401	20.03	.6712	.2131	21.31	....	Normal Consistency, Neat
7000	6404	**3.500**	4.118	.3560	2.663	22.21	.7444	.2363	23.63	....	
6500	5891	**3.250**	3.788	.3940	2.947	24.59	.8239	.2615	26.15	....	
6000	5378	**3.000**	3.459	.4354	3.257	27.17	.9104	.2890	28.90	....	
5800	5177	2.900	3.329	.4528	3.387	28.25	.9468	.3006	30.06	7.516	
5600	4975	2.800	3.199	.4709	3.522	29.38	.9847	.3126	31.26	7.817	
5500	4876	**2.750**	3.136	.4800	3.590	29.95	1.004	.3186	31.86	7.968	
5400	4775	2.700	3.071	.4896	3.662	30.55	1.024	.3250	32.50	8.127	
5200	4575	2.600	2.942	.5096	3.807	31.76	1.064	.3379	33.79	8.449	
5000	4378	**2.500**	2.815	.5291	3.958	33.02	1.106	.3512	35.12	8.783	
4800	4180	2.400	2.688	.5501	4.115	34.33	1.150	.3652	36.52	9.132	
4600	3984	2.300	2.562	.5720	4.279	35.69	1.196	.3797	37.97	9.495	
4500	3889	**2.250**	2.501	.5830	4.361	36.38	1.219	.3870	38.70	9.678	
4400	3789	2.200	2.437	.5948	4.449	37.12	1.244	.3948	39.48	9.873	1:3 Standard Sand
4200	3595	2.100	2.312	.6187	4.628	38.61	1.294	.4107	41.07	10.27	
4000	3402	**2.000**	2.188	.6438	4.816	40.17	1.346	.4274	42.74	10.69	
3800	3211	1.900	2.065	.6701	5.012	41.81	1.401	.4448	44.48	11.12	
3600	3021	1.800	1.943	.6979	5.220	43.55	1.459	.4633	46.33	11.59	
3500	2929	**1.750**	1.884	.7120	5.326	44.43	1.489	.4726	47.26	11.82	Usual Mortars, Stiff or Rich Concretes
3400	2832	1.700	1.821	.7273	5.440	45.38	1.521	.4828	48.28	12.07	
3250	2694	**1.625**	1.732	.7500	5.610	46.80	1.568	.4979	49.79	12.45	
3200	2644	1.600	1.700	.7585	5.674	47.33	1.586	.5035	50.35	12.59	
3000	2459	**1.500**	1.581	.7916	5.921	49.40	1.655	.5255	52.55	13.14	
2800	2275	1.400	1.463	.8270	6.186	51.60	1.729	.5490	54.90	13.73	
2750	2230	**1.375**	1.434	.8360	6.253	52.17	1.748	.5549	55.49	13.88	
2600	2092	1.300	1.345	.8652	6.472	53.99	1.809	.5743	57.43	14.36	
2500	2000	**1.250**	1.286	.8850	6.620	55.22	1.851	.5875	58.75	14.69	
2400	1911	1.200	1.229	.9063	6.779	56.55	1.895	.6016	60.16	15.04	
2250	1783	**1.125**	1.147	.9380	7.016	58.53	1.961	.6226	62.26	15.57	Usual Concretes
2200	1732	1.100	1.114	.9510	7.113	59.34	1.989	.6313	63.13	15.79	
2000	**1555**	**1.000**	**1.000**	**1.000**	**7.480**	**62.40**	**2.091**	**.6638**	**66.38**	**16.60**	
1800	1383	.9000	.8894	1.054	7.884	65.77	2.204	.7000	70.00	17.50	
1750	1340	**.8750**	.8617	1.068	7.989	66.64	2.233	.7089	70.89	17.73	
1600	1208	.8000	.7768	1.115	8.340	69.58	2.331	.7401	74.01	18.51	
1500	1126	**.7500**	.7241	1.147	8.580	71.57	2.398	.7614	76.14	19.04	
1400	1041	.7000	.6695	1.183	8.849	73.82	2.474	.7853	78.53	19.64	
1250	914	**.6250**	.5878	1.242	9.290	77.50	2.597	.8244	82.44	20.62	
1200	873	.6000	.5614	1.263	9.447	78.81	2.641	.8384	83.84	20.97	
1000	712	**.5000**	.4579	1.356	10.14	84.61	2.835	.9001	90.01	22.51	
900	632	.4500	.4064	1.410	10.55	87.98	2.948	.9360	93.60	23.41	
800	553	.4000	.3556	1.471	11.00	91.79	3.076	.9764	97.64	24.42	
750	515	**.3750**	.3312	1.503	11.24	93.79	3.143	.9977	99.77	24.95	Wet or Lean Concretes
700	476	.3500	.3061	1.539	11.51	96.03	3.218	1.022	102.2	25.55	
600	399	.3000	.2566	1.619	12.11	101.0	3.385	1.075	107.5	26.88	
500	325	**.2500**	.2090	1.712	12.81	106.8	3.580	1.136	113.6	28.42	
400	253	.2000	.1627	1.827	13.67	114.0	3.820	1.213	121.3	30.33	
300	183	**.1500**	.1177	1.975	14.77	123.2	4.130	1.311	131.1	32.79	
200	116	.1000	.0746	2.183	16.63	136.2	4.565	1.449	144.9	36.24	

TABLE IV.—ESTIMATED STRENGTHS AND STRENGTH RATIOS FOR AVERAGE JOB, USING $S = \frac{14{,}000}{9^x}$ (ABRAMS)

Strength, 28-Day Standard, Approximate (Abrams)		Ratio of Strength to that for W/C = 1		Ratio of Water to Cement Expressed in Different Recognized Units							General
Rigid Control $S = \frac{14{,}000}{7x}$	Average Job $S = \frac{14{,}000}{9x}$	$S = \frac{14{,}000}{7x}$	$S = \frac{14{,}000}{9x}$	W/C by Loose Volume (Abrams)	Gallons per Bag of Cement	Lb. per Bag of Cement	W/C by Absolute Volume (Talbot)	W/C by Weight (lb./lb.; l./kg.; c.c./g.; etc.)	Per Cent Cement by Weight (100 × Col. 9)	Per Cent Weight Cement plus Sand in 1:3 Mixtures	Description and Remarks
1	2	3	4	5	6	7	8	9	10	11	12
Strength		Strength Ratios									
Rigid	Aver. Job	Rigid	Aver. Job	W/C (Abrams)	Gal. per Bag	Lb. per Bag	W/C Absolute (Talbot)	W/C by Wt.	Per Cent by Wt.	1:3 Per Cent Total Wt.	
9000	**8500**	4.500	5.468	.2269	1.697	14.16	.4743	.1506	15.06	....	Normal Consistency, Neat
8529	**8000**	4.265	5.145	.2547	1.905	15.89	.5326	.1691	16.91	....	
8056	**7500**	4.028	4.823	.2840	2.124	17.72	.5938	.1885	18.85	....	
7580	**7000**	3.790	4.502	.3153	2.358	19.67	.6593	.2093	20.93	....	
7099	**6500**	3.550	4.180	.3490	2.611	21.78	.7298	.2317	23.17	....	
6611	**6000**	3.306	3.859	.3856	2.884	24.06	.8063	.2560	25.60	....	
6123	**5500**	3.062	3.537	.4250	3.179	26.52	.8887	.2821	28.21	....	
5625	**5000**	2.813	3.215	.4686	3.505	29.24	.9798	.3111	31.11	7.779	
5129	**4500**	2.565	2.894	.5160	3.860	32.20	1.079	.3425	34.25	8.566	
4616	**4000**	2.308	2.572	.5702	4.265	35.58	1.192	.3785	37.85	9.465	Usual Mortars, Stiff or Rich Concretes
4103	**3500**	2.052	2.251	.6307	4.718	39.36	1.319	.4187	41.87	10.47	1:3 Std. Sand
3844	**3250**	1.922	2.090	.6642	4.968	41.45	1.389	.4409	44.09	11.03	
3578	**3000**	1.789	1.929	.7011	5.244	43.75	1.466	.4654	46.54	11.64	
3311	**2750**	1.656	1.768	.7410	5.543	46.24	1.549	.4919	49.19	12.30	
3045	**2500**	1.523	1.608	.7840	5.864	48.92	1.639	.5204	52.04	13.01	
2776	**2250**	1.388	1.447	.8315	6.220	51.89	1.739	.5519	55.19	13.80	Usual Concretes
2500	**2000**	1.250	1.286	.8850	6.620	55.22	1.851	.5875	58.75	14.69	
2222	**1750**	1.111	1.125	.9460	7.076	59.03	1.978	.6280	62.80	15.70	
2000	**1555**	**1.000**	**1.000**	**1.000**	**7.480**	**62.40**	**2.091**	**.6638**	**66.38**	**16.60**	
1935	**1500**	.9675	.9646	1.017	7.607	63.46	2.127	.6751	67.51	16.88	
1650	**1250**	.8250	.8039	1.099	8.221	68.58	2.298	.7295	72.95	18.24	
1353	**1000**	.6765	.6431	1.201	8.983	74.94	2.511	.7972	79.72	19.94	Wet or Lean Concretes
1050	**750**	.5250	.4823	1.331	9.956	83.05	2.783	.8835	88.35	22.09	
731	**500**	.3655	.3215	1.517	11.35	94.66	3.172	1.007	100.7	25.18	
466	**300**	.2330	.1929	1.749	13.08	109.1	3.657	1.161	116.1	29.03	

TABLE V.—WATER-CEMENT RATIO BY LOOSE VOLUME (ABRAMS)

Strength, 28-Day Standard, Approximate (Abrams)		Ratio of Strength to that for W/C = 1		Ratio of Water to Cement Expressed in Different Recognized Units							General
Rigid Control $S = \frac{14,000}{7x}$	Average Job $S = \frac{14,000}{9x}$	$S = \frac{14,000}{7x}$	$S = \frac{14,000}{9x}$	W/C by Loose Volume (Abrams)	Gallons per Bag of Cement	Lb. per Bag of Cement	W/C by Absolute Volume (Talbot)	W/C by Weight (lb./lb.; l./kg.; c.c./g.; etc.)	Per Cent Cement by Weight (100 × Col. 9)	Per Cent Weight Cement plus Sand in 1:3 Mixtures	Description and Remarks
1	2	3	4	5	6	7	8	9	10	11	12
Strength		Strength Ratios									
Rigid	Aver. Job	Rigid	Aver. Job	W/C (Abrams)	Gal. per Bag	Lb. per Bag	W/C Absolute (Talbot)	W/C by Wt.	Per Cent by Wt.	1:3 Per Cent Total Wt.	
8607	8083	4.304	5.198	.2500	1.870	15.60	.5228	.1660	16.60	....	Normal Consistency Neat
7827	7242	3.914	4.657	.3000	2.244	18.72	.6273	.1991	19.91	....	
7085	6489	3.543	4.173	.3500	2.618	21.84	.7319	.2323	23.23	....	
6428	5813	3.214	3.738	.4000	2.992	24.96	.8364	.2655	26.55	....	
5832	5209	2.916	3.350	.4500	3.366	28.08	.9410	.2987	29.87	....	
5292	4667	2.646	3.001	.5000	3.740	31.20	1.046	.3319	33.19	8.300	1:3 Std. Sand
4812	4181	2.406	2.689	.5500	4.114	34.32	1.150	.3651	36.51	9.130	
4356	3746	2.178	2.409	.6000	4.488	37.44	1.255	.3983	39.83	9.960	
3952	3356	1.976	2.158	.6500	4.862	40.56	1.359	.4315	43.15	10.79	Usual Mortars, Stiff or Rich Concretes
3586	3007	1.793	1.934	.7000	5.236	43.68	1.464	.4647	46.47	11.62	
3250	2694	1.625	1.732	.7500	5.610	46.80	1.568	.4979	49.79	12.45	
2952	2414	1.476	1.552	.8000	5.984	49.92	1.673	.5310	53.10	13.28	Usual Concretes
2678	2163	1.339	1.391	.8500	6.358	53.04	1.777	.5642	56.42	14.11	
2430	1938	1.215	1.246	.9000	6.732	56.16	1.882	.5974	59.74	14.94	
2204	1736	1.102	1.116	.9500	7.106	59.28	1.986	.6306	63.06	15.77	
2000	**1555**	**1.000**	**1.000**	**1.000**	**7.480**	**62.40**	**2.091**	**.6638**	**66.38**	**16.60**	
1815	1394	.9075	.8965	1.050	7.854	65.52	2.196	.6970	69.70	17.43	
1646	1249	.8230	.8032	1.100	8.228	68.64	2.300	.7302	73.02	18.26	
1494	1119	.7470	.7196	1.150	8.602	71.76	2.405	.7634	76.34	19.09	
1355	1002	.6775	.6444	1.200	8.976	74.88	2.509	.7966	79.66	19.92	
1230	898	.6150	.5775	1.250	9.350	78.00	2.614	.8298	82.98	20.75	Wet or Lean Concretes
1116	805	.5580	.5177	1.300	9.724	81.12	2.718	.8629	86.29	21.58	
1012	721	.5060	.4637	1.350	10.10	84.24	2.823	.8961	89.61	22.41	
918	646	.4590	.4154	1.400	10.47	87.36	2.927	.9293	92.93	23.24	
833	579	.4165	.3723	1.450	10.85	90.48	3.032	.9625	96.25	24.07	
756	519	.3780	.3338	1.500	11.22	93.60	3.137	.9957	99.57	24.90	
686	465	.3430	.2990	1.550	11.59	96.72	3.241	1.029	102.9	25.73	
622	416	.3110	.2675	1.600	11.97	99.84	3.346	1.062	106.2	26.56	
565	373	.2825	.2399	1.650	12.34	103.0	3.450	1.095	109.5	27.39	
512	334	.2560	.2148	1.700	12.72	106.1	3.555	1.128	112.8	28.22	
465	299	.2325	.1923	1.750	13.09	109.2	3.659	1.162	116.2	29.05	
422	268	.2110	.1723	1.800	13.46	112.3	3.764	1.195	119.5	29.88	
383	240	.1915	.1543	1.850	13.84	115.4	3.868	1.228	122.8	30.71	
347	215	.1735	.1383	1.900	14.21	118.6	3.973	1.261	126.1	31.54	
315	193	.1575	.1241	1.950	14.59	121.7	4.077	1.294	129.4	32.37	
286	173	.1430	.1113	2.000	14.96	124.8	4.182	1.328	132.8	33.20	

TABLE VI.—WATER-CEMENT RATIO BY GALLONS PER BAG OF CEMENT

Strength, 28-Day Standard, Approximate (Abrams): Rigid Control $S = \frac{14,000}{7x}$	Average Job $S = \frac{14,000}{9x}$	Ratio of Strength to that for W/C = 1: $S = \frac{14,000}{7x}$	$S = \frac{14,000}{9x}$	Ratio of Water to Cement Expressed in Different Recognized Units: W/C by Loose Volume (Abrams)	Gallons per Bag of Cement	Lb. per Bag of Cement	W/C by Absolute Volume (Talbot)	W/C by Weight (lb./lb.; l./kg.; c.c./g.; etc.)	Per Cent Cement by Weight (100 × Col. 9)	Per Cent Weight Cement plus Sand in 1:3 Mixtures	General: Description and Remarks
1	2	3	4	5	6	7	8	9	10	11	12
Strength: Rigid	Aver. Job	Strength Ratios: Rigid	Aver. Job	W/C (Abrams)	Gal. per Bag	Lb. per Bag	W/C Absolute (Talbot)	W/C by Wt.	Per Cent by Wt.	1:3 Per Cent Total Wt.	
8320	7780	4.160	5.003	.2674	**2.000**	16.69	.5591	.1775	17.75	....	
7305	6716	3.653	4.319	.3343	**2.500**	20.86	.6990	.2219	22.19	....	Normal Consistency, Neat
6414	5800	3.207	3.729	.4011	**3.000**	25.03	.8387	.2663	26.63	....	
5631	5007	2.816	3.220	.4680	**3.500**	29.20	.9786	.3107	31.07	....	
4945	4323	2.473	2.780	.5348	**4.000**	33.37	1.118	.3550	35.50	8.878	
4341	3732	2.171	2.400	.6017	**4.500**	37.55	1.258	.3994	39.94	9.988	1:3 Std. Sand
3812	3223	1.906	2.073	.6685	**5.000**	41.71	1.398	.4440	44.40	11.10	
3347	2782	1.674	1.789	.7354	**5.500**	45.89	1.538	.4882	48.82	12.21	Usual Mortars, Stiff or Rich Concretes
2939	2402	1.470	1.545	.8022	**6.000**	50.06	1.677	.5325	53.25	13.32	
2580	2074	1.290	1.334	.8691	**6.500**	54.23	1.817	.5769	57.69	14.43	
2266	1792	1.133	1.152	.9359	**7.000**	58.40	1.957	.6213	62.13	15.54	
1989	**1546**	**.9945**	**.9942**	**1.003**	**7.500**	**62.57**	**2.097**	**.6657**	**66.57**	**16.65**	Usual Concretes
1745	1334	.8725	.8579	1.070	**8.000**	66.77	2.237	.7103	71.03	17.76	
1532	1151	.7660	.7402	1.137	**8.500**	70.95	2.377	.7547	75.47	18.87	
1347	996	.6735	.6405	1.203	**9.000**	75.07	2.515	.7986	79.86	19.97	
1183	860	.5915	.5531	1.270	**9.500**	79.25	2.656	.8430	84.30	21.08	
1038	742	.5190	.4772	1.337	**10.00**	83.43	2.796	.8875	88.75	22.19	
911	640	.4555	.4116	1.404	**10.50**	87.61	2.936	.9320	93.20	23.31	
800	553	.4000	.3556	1.471	**11.00**	91.79	3.076	.9764	97.64	24.42	
702	477	.3510	.3068	1.538	**11.50**	95.97	3.216	1.021	102.1	25.53	
617	413	.3085	.2656	1.604	**12.00**	100.1	3.354	1.065	106.5	26.63	Wet or Lean Concretes.
542	356	.2710	.2289	1.671	**12.50**	104.3	3.494	1.109	110.9	27.74	
476	307	.2380	.1974	1.738	**13.00**	108.5	3.634	1.154	115.4	28.85	
418	265	.2090	.1704	1.805	**13.50**	112.6	3.774	1.198	119.8	29.96	
367	229	.1835	.1473	1.872	**14.00**	116.8	3.914	1.243	124.3	31.08	
322	198	.1610	.1273	1.939	**14.50**	121.0	4.054	1.287	128.7	32.19	
282	171	.1410	.1100	2.006	**15.00**	125.2	4.195	1.332	133.2	33.30	

TABLE VII.—WATER-CEMENT RATIO BY POUNDS OF WATER PER BAG OF CEMENT

Strength, 28-Day Standard, Approximate (Abrams)		Ratio of Strength to that for W/C = 1		Ratio of Water to Cement Expressed in Different Recognized Units							General
Rigid Control $S = \frac{14,000}{7^x}$	Average Job $S = \frac{14,000}{9^x}$	$S = \frac{14,000}{7^x}$	$S = \frac{14,000}{9^x}$	W/C by Loose Volume (Abrams)	Gallons per Bag of Cement	Lb. per Bag of Cement	W/C by Absolute Volume (Talbot)	W/C by Weight (lb./lb.; l./kg.; c.c./g.; etc.)	Per Cent Cement by Weight (100 × Col. 9)	Per Cent Weight Cement plus Sand in 1:3 Mixtures	Description and Remarks
1	2	3	4	5	6	7	8	9	10	11	12
Strength		Strength Ratios									
Rigid	Aver. Job	Rigid	Aver. Job	W/C (Abrams)	Gal. per Bag	Lb. per Bag	W/C Absolute (Talbot)	W/C by Wt.	Per Cent by Wt.	1:3 Per Cent Total Wt.	
8768	8254	4.384	5.308	.2405	1.799	**15.00**	.5029	.1596	15.96	....	Normal Consistency, Neat
7502	6922	3.751	4.451	.3206	2.398	**20.00**	.6704	.2128	21.28	....	
6418	5803	3.209	3.732	.4008	2.998	**25.00**	.8381	.2661	26.61	....	
5492	4867	2.746	3.130	.4809	3.597	**30.00**	1.006	.3192	31.92	7.983	
4698	4080	2.349	2.624	.5611	4.197	**35.00**	1.173	.3725	37.25	9.314	1:3 Std. Sand
4020	3422	2.010	2.201	.6412	4.796	**40.00**	1.341	.4256	42.56	10.64	Usual Mortars, Stiff or Rich Concretes
3439	2869	1.720	1.845	.7214	5.396	**45.00**	1.508	.4789	47.89	11.98	
2943	2406	1.472	1.547	.8015	5.995	**50.00**	1.676	.5320	53.20	13.30	
2512	2017	1.256	1.297	.8817	6.595	**55.00**	1.844	.5853	58.53	14.64	Usual Concretes
2159	1692	1.080	1.088	.9618	7.194	**60.00**	2.011	.6384	63.84	15.97	
1843	1418	.9215	.9119	1.042	7.794	**65.00**	2.179	.6917	69.17	17.30	
1588	1190	.7940	.7653	1.122	8.393	**70.00**	2.346	.7448	74.48	18.63	
1350	998	.6750	.6418	1.202	8.991	**75.00**	2.513	.7979	79.79	19.95	
1155	837	.5775	.5383	1.282	9.589	**80.00**	2.681	.8509	85.09	21.28	Wet or Lean Concretes
987	700	.4935	.4508	1.363	10.20	**85.00**	2.850	.9048	90.48	22.63	
845	588	.4225	.3781	1.443	10.79	**90.00**	3.017	.9579	95.79	23.95	
723	493	.3615	.3170	1.523	11.39	**95.00**	3.185	1.011	101.1	25.28	
619	414	.3095	.2662	1.603	11.99	**100.0**	3.352	1.064	106.4	26.61	
530	347	.2650	.2232	1.683	12.59	**105.0**	3.519	1.117	111.7	29.94	
453	291	.2265	.1871	1.763	13.19	**110.0**	3.686	1.170	117.0	29.27	
388	244	.1940	.1569	1.843	13.79	**115.0**	3.854	1.223	122.3	30.59	
331	204	.1655	.1312	1.924	14.39	**120.0**	4.023	1.277	127.7	31.94	
284	171	.1420	.1100	2.004	14.99	**125.0**	4.190	1.330	133.0	33.27	

TABLE VIII.—WATER-CEMENT RATIO BY ABSOLUTE VOLUME

Strength, 28-Day Standard, Approximate (Abrams)		Ratio of Strength to that for W/C = 1		Ratio of Water to Cement Expressed in Different Recognized Units							General
Rigid Control $S = \frac{14,000}{7x}$	Average Job $S = \frac{14,000}{9x}$	$S = \frac{14,000}{7x}$	$S = \frac{14,000}{9x}$	W/C by Loose Volume (Abrams)	Gallons per Bag of Cement	Lb. per Bag of Cement	W/C by Absolute Volume (Talbot)	W/C by Weight (lb./lb.; l./kg.; c.c./g.; etc.)	Per Cent Cement by Weight (100 × Col. 9)	Per Cent Weight Cement plus Sand in 1:3 Mixtures	Description and Remarks
1	2	3	4	5	6	7	8	9	10	11	12
Strength		Strength Ratios									
Rigid	Aver. Job	Rigid	Aver. Job	W/C (Abrams)	Gal. per Bag	Lb. per Bag	W/C Absolute (Talbot)	W/C by Wt.	Per Cent by Wt.	1:3 Per Cent Total Wt.	
8011	7454	4.006	4.794	.2869	2.146	17.90	**0.600**	.1904	19.04	....	Normal Consistency, Neat
6650	6040	3.325	3.884	.3826	2.862	23.87	**0.800**	.2540	25.40	....	
5521	4896	2.762	3.149	.4782	3.577	29.84	**1.000**	.3174	31.74	7.936	
4584	3968	2.292	2.552	.5738	4.292	35.81	**1.200**	.3809	38.09	9.525	1:3 Std. Sand
3805	3216	1.903	2.068	.6695	5.008	41.78	**1.400**	.4444	44.44	11.11	
3159	2606	1.579	1.676	.7651	5.723	47.74	**1.600**	.5079	50.79	12.70	Usual Mortars, Stiff or Rich Concretes
2622	2112	1.311	1.358	.8608	6.439	53.71	**1.800**	.5714	57.14	14.29	Usual Concretes
2177	1712	1.089	1.101	.9564	7.154	59.70	**2.000**	.6349	63.49	15.88	
1808	1388	.9040	.8926	1.052	7.869	65.64	**2.200**	.6983	69.83	17.46	
1499	1124	.7495	.7228	1.148	8.587	71.64	**2.400**	.7620	76.20	19.06	
1246	912	.6230	.5865	1.243	9.298	77.56	**2.600**	.8251	82.51	20.63	Wet or Lean Concretes
1034	739	.5170	.4752	1.339	10.02	83.55	**2.800**	.8888	88.88	22.23	
858	598	.4290	.3846	1.435	10.73	89.54	**3.000**	.9526	95.26	23.82	
713	485	.3565	.3119	1.530	11.44	95.47	**3.200**	1.016	101.6	25.40	
592	393	.2960	.2527	1.626	12.16	101.5	**3.400**	1.079	107.9	26.99	
491	318	.2455	.2045	1.722	12.88	107.5	**3.600**	1.143	114.3	28.59	
408	258	.2040	.1659	1.817	13.59	113.4	**3.800**	1.206	120.6	30.16	
338	209	.1690	.1344	1.913	14.31	119.4	**4.000**	1.270	127.0	31.76	
281	170	.1405	.1093	2.008	15.02	125.3	**4.200**	1.333	133.3	33.33	

TABLE IX.—WATER-CEMENT RATIO BY WEIGHT AND AS PERCENTAGE BY WEIGHT

Strength, 28-Day Standard, Approximate (Abrams)		Ratio of Strength to that for W/C = 1		Ratio of Water to Cement Expressed in Different Recognized Units							General
Rigid Control $S = \frac{14,000}{7^x}$	Average Job $S = \frac{14,000}{9^x}$	$S = \frac{14,000}{7^x}$	$S = \frac{14,000}{9^x}$	W/C by Loose Volume (Abrams)	Gallons per Bag of Cement	Lb. per Bag of Cement	W/C by Absolute Volume (Talbot)	W/C by Weight (lb./lb.; l./kg.; c.c./g.; etc.)	Per Cent Cement by Weight (100 × Col. 9)	Per Cent Weight Cement plus Sand in 1:3 Mixtures	Description and Remarks
1	2	3	4	5	6	7	8	9	10	11	12
Strength		Strength Ratios									
Rigid	Aver. Job	Rigid	Aver. Job	W/C (Abrams)	Gal. per Bag	Lb. per Bag	W/C Absolute (Talbot)	W/C by Wt.	Per Cent by Wt.	1:3 Per Cent Total Wt.	
9020	8523	4.560	5.481	.2259	1.690	14.10	.4724	**.1500**	**15.00**	....	Normal Consistency, Neat
7791	7223	3.896	4.645	.3012	2.253	18.79	.6298	**.2000**	**20.00**	....	
6729	6122	3.365	3.937	.3765	2.816	23.49	.7873	**.2500**	**25.00**	....	
5812	5188	2.906	3.336	.4518	3.379	28.19	.9447	**.3000**	**30.00**	....	
5020	4397	2.510	2.828	.5271	3.943	32.89	1.102	**.3500**	**35.00**	8.750	
4336	3727	2.168	2.397	.6024	4.506	37.59	1.260	**.4000**	**40.00**	10.00	
4089	**3488**	**2.045**	**2.243**	**.6325**	**4.731**	**39.47**	**1.323**	**.4200**	**42.00**	**10.50**	1:3 Std. Sand; Usual Mortars, Stiff or Rich Concretes
3745	3158	1.873	2.031	.6777	5.069	42.29	1.417	**.4500**	**45.00**	11.25	
3234	2677	1.617	1.722	.7530	5.632	46.99	1.575	**.5000**	**50.00**	12.50	
2793	2268	1.397	1.459	.8283	6.196	51.69	1.732	**.5500**	**55.00**	13.75	
2413	1923	1.207	1.237	.9036	6.759	56.38	1.889	**.6000**	**60.00**	15.00	Usual Concretes
2084	1630	1.042	1.048	.9789	7.322	61.08	2.047	**.6500**	**65.00**	16.25	
1800	1383	.9000	.8894	1.054	7.884	65.77	2.204	**.7000**	**70.00**	17.50	
1555	1169	.7775	.7518	1.130	8.452	70.51	2.363	**.7500**	**75.00**	18.76	
1342	991	.6710	.6373	1.205	9.013	75.19	2.520	**.8000**	**80.00**	20.00	Wet or Lean Concretes
1160	841	.5800	.5408	1.280	9.574	79.87	2.676	**.8500**	**85.00**	21.25	
1002	713	.5010	.4585	1.355	10.14	84.55	2.833	**.9000**	**90.00**	22.49	
865	603	.4325	.3878	1.431	10.70	89.29	2.992	**.9500**	**95.00**	23.75	
747	512	.3735	.3293	1.506	11.26	94.00	3.149	**1.000**	**100.0**	24.98	
646	434	.3230	.2791	1.581	11.83	98.65	3.306	**1.050**	**105.0**	26.24	
557	367	.2785	.2360	1.657	12.39	103.4	3.465	**1.100**	**110.0**	27.51	
481	311	.2405	.2000	1.732	12.96	108.1	3.622	**1.150**	**115.0**	28.75	
416	264	.2080	.1698	1.807	13.52	112.8	3.778	**1.200**	**120.0**	30.00	
359	224	.1795	.1441	1.883	14.08	117.5	3.937	**1.250**	**125.0**	31.26	
310	190	.1550	.1222	1.958	14.65	122.2	4.094	**1.300**	**130.0**	32.50	
268	161	.1340	.1035	2.033	15.21	126.9	4.251	**1.350**	**135.0**	33.75	

Table X.—Normal Consistency Range

Strength, 28-Day Standard, Approximate (Abrams)		Ratio of Strength to that for W/C = 1		Ratio of Water to Cement Expressed in Different Recognized Units							General		
1	2	3	4	5	6	7	8	9	10	11	12	13	
Strength		Strength Ratios											
Rigid	Aver. Job	Rigid	Aver. Job	W/C (Abrams)	Gal. per Bag	Lb. per Bag	W/C Absolute (Talbot)	W/C by Wt.	Per Cent by Wt.	1:3 Per Cent Total Wt.			
9020	8523	4.560	5.481	.2259	1.690	14.10	.4724	.150	15.0	1:3 Not Workable	Normal Consistency Range, Neat Cements	9.0	H_2O to be Used for 1:3 Standard Sand Mixture at this Normal Consistency (Col. 10)
8759	8244	4.380	5.302	.2410	1.803	15.04	.5039	.160	16.0			9.2	
8500	7975	4.250	5.129	.2561	1.916	15.98	.5355	.170	17.0			9.3	
8262	7719	4.131	4.964	.2710	2.027	16.91	.5667	.180	18.0			9.5	
8020	7464	4.010	4.800	.2863	2.142	17.87	.5987	.190	19.0			9.7	
7791	7223	3.896	4.645	.3012	2.253	18.79	.6298	.200	20.0			9.8	
7565	6987	3.783	4.493	.3163	2.366	19.74	.6614	.210	21.0			10.0	
7348	6761	3.674	4.347	.3313	2.478	20 67	.6927	.220	22.0			10.2	
7135	6540	3.568	4.206	.3464	2.591	21.62	.7243	.230	23.0			10.3	
6930	6328	3.465	4.069	.3614	2.703	22.55	.7557	.240	24.0			10.5	
6729	6122	3.365	3.937	.3765	2.816	23.49	.7873	.250	25.0			10.7	
6534	5922	3.267	3.808	.3916	2.929	24.44	.8188	.260	26.0			10.8	
6346	5730	3.173	3.685	.4066	3.041	25.37	.8502	.270	27.0			11.0	
6162	5543	3.081	3.565	.4217	3.154	26.31	.8818	.280	28.0			11.2	
5985	5363	2.993	3.449	.4367	3.267	27.25	.9131	.290	29.0			11.3	
5812	5188	2.906	3.336	.4518	3.379	28.19	.9447	.300	30.0			11.5	
5644	5019	2.822	3.228	.4669	3.492	29.13	.9763	.310	31.0	7.75	.....	....	
5481	4856	2.741	3.123	.4819	3.605	30.07	1.008	.320	32.0	8.00	.....	....	
5323	4698	2.662	3.021	.4970	3.718	31.01	1.039	.330	33.0	8.25	.....	....	
5169	4545	2.585	2.923	.5120	3.830	31.95	1.071	.340	34.0	8.50	.....	....	
5020	4397	2.510	2.828	.5271	3.943	32.89	1.102	.350	35.0	8.75	.....	....	
4874	4253	2.437	2.735	.5422	4.056	33.83	1.134	.360	36.0	9.00	Normal Consistency Range for 1:3 Standard Sand Mortars	15.0	Normal Consistency of Neat Cement Corresponding to this Water Requirement for 1:3 Standard Sand Mortar (Col. 11)
4846	4226	2.423	2.718	.5452	4.078	34.02	1.140	.362	36.2	9.05		....	
4818	4198	2.409	2.700	.5482	4.101	34.21	1.146	.364	36.4	9.10		....	
4790	4170	2.395	2.682	.5512	4.123	34.39	1.153	.366	36.6	9.15		....	
4762	4143	2.381	2.664	.5542	4.145	34.58	1.159	.368	36.8	9.20		16.0	
4734	4116	2.367	2.647	.5572	4.168	34.77	1.165	.370	37.0	9.25		....	
4707	4089	2.354	2.630	.5602	4.190	34.96	1.171	.372	37.2	9.30		17.0	
4679	4062	2.340	2.612	.5632	4.213	35.14	1.178	.374	37.4	9.35		....	
4652	4035	2.326	2.595	.5662	4.235	35.33	1.184	.376	37.6	9.40		....	
4625	4008	2.313	2.577	.5692	4.258	35.52	1.190	378	37.8	9.50		....	
4597	3981	2.299	2.560	.5723	4.281	35.71	1.197	.380	38.0	9.50	Stiff or Rich Concretes—Usual Mortars	18.0	
4570	3955	2.285	2.543	.5753	4.303	35.90	1.203	.382	38.2	9.55		....	
4544	3929	2.272	2.527	.5783	4.326	36.09	1.209	.384	38.4	9.60		....	
4517	3903	2.258	2.510	.5813	4.348	36.27	1.215	.386	38.6	9.65		....	
4491	3878	2.246	2.494	.5843	4.371	36.46	1.222	.388	38.8	9.70		19.0	
4465	3852	2.233	2.477	.5873	4.393	36.65	1.228	.390	39.0	9.75		....	
4439	3827	2.220	2.461	.5903	4.415	36.83	1.234	.392	39.2	9.80		20.0	
4413	3802	2.207	2.445	.5933	4.438	37.02	1.241	.394	39.4	9.85		....	
4387	3777	2.194	2.429	.5963	4.460	37.21	1.247	.396	39.6	9.90		....	
4362	3752	2.181	2.413	.5993	4.483	37.40	1.253	.398	39.8	9.95		....	
4336	3727	2.168	2.397	.6024	4.506	37.59	1.260	.400	40.0	10.0		21.0	
4310	3702	2.155	2.381	.6054	4.528	37.78	1.266	.402	40.2	10.05		....	
4285	3678	2.143	2.365	.6084	4.550	37.96	1.273	.404	40.4	10.1		....	
4260	3654	2.130	2.350	.6114	4.573	38.15	1.278	.406	40.6	10.15		....	
4235	3629	2.118	2.334	.6144	4.596	38.34	1.285	.408	40.8	10.2		22.0	
4210	3605	2.105	2.318	.6175	4.619	38.53	1.291	.410	41.0	10.25		....	
4186	3581	2.093	2.303	.6205	4.641	38.72	1.297	.412	41.2	10.3		23.0	
4161	3558	2.081	2.288	.6235	4.664	38.91	1.304	.414	41.4	10.35		....	
4137	3534	2.068	2.273	.6265	4.686	39.09	1.310	.416	41.6	10.4		....	
4113	3511	2.057	2.258	.6295	4.709	39.28	1.316	.418	41.8	10.45		....	
4089	3488	2.045	2.243	.6325	4.731	39.47	1.323	.420	42.0	10.5		24.0	
4065	3465	2.033	2.228	.6355	4.754	39.66	1.329	.422	42.2	10.55		....	
4041	3442	2.021	2.214	.6385	4.776	39.84	1.335	.424	42.4	10.6		....	
4018	3420	2.009	2.199	.6415	4.798	40.03	1.341	.426	42.6	10.65		....	
3994	3397	1.997	2.185	.6445	4.821	40.22	1.348	.428	42.8	10.7		25.0	
3970	3374	1.985	2.170	.6476	4.843	40.41	1.354	.430	43.0	10.75		....	
3947	3352	1.974	2.156	.6506	4.866	40.60	1.360	.432	43.2	10.8		26.0	
3924	3330	1.962	2.141	.6536	4.889	40.78	1.367	.434	43.4	10.85		....	
3902	3308	1.951	2.127	.6566	4.911	40.97	1.373	.436	43.6	10.9		....	
3879	3286	1.940	2.113	.6596	4.934	41.16	1.379	.438	43.8	10.95		....	
3856	3265	1.928	2.100	.6626	4.956	41.35	1.386	.440	44.0	11.0		27.0	
3834	3243	1.917	2.086	.6656	4.979	41.53	1.392	.442	44.2	11.05		....	
3812	3223	1.906	2.073	.6685	5.000	41.71	1.398	.444	44.4	11.1		....	
3789	3200	1.895	2.058	.6717	5.024	41.91	1.405	.446	44.6	11.15		....	
3767	3180	1.884	2.045	.6746	5.046	42.10	1.411	.448	44.8	11.2		28.0	
3745	3158	1.873	2.031	.6777	5.069	42.29	1.417	.450	45.0	11.25		....	
3723	3138	1.862	2.018	.6807	5.092	42.48	1.423	.452	45.2	11.3		29.0	
3701	3117	1.851	2.005	.6837	5.114	42.66	1.430	.454	45.4	11.35		....	
3680	3096	1.840	1.991	.6867	5.137	42.85	1.436	.456	45.6	11.4		....	
3658	3076	1.829	1.978	.6897	5.159	43.04	1.442	.458	45.8	11.45		....	
3637	3056	1.818	1.965	.6927	5.181	43.22	1.448	.460	46.0	11.5		30.0	

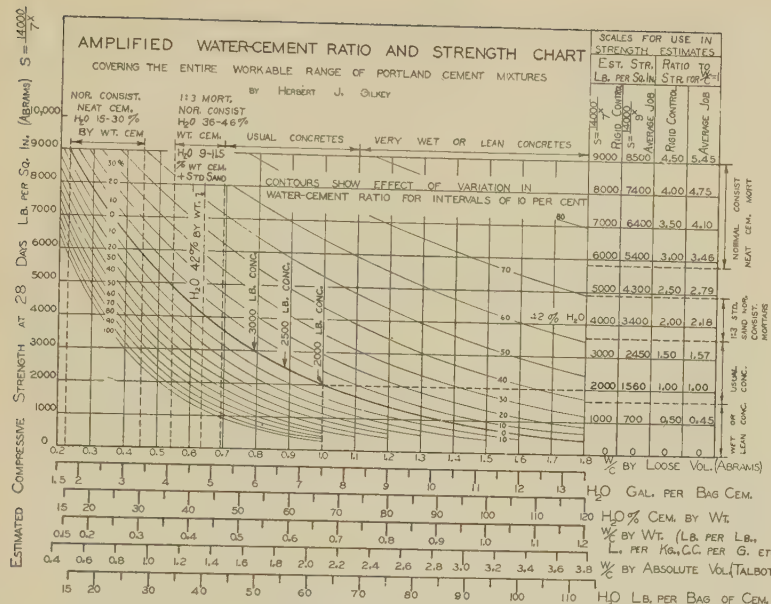

FIG. 1—COMPLETE WATER-CEMENT RATIO AND STRENGTH CHART.

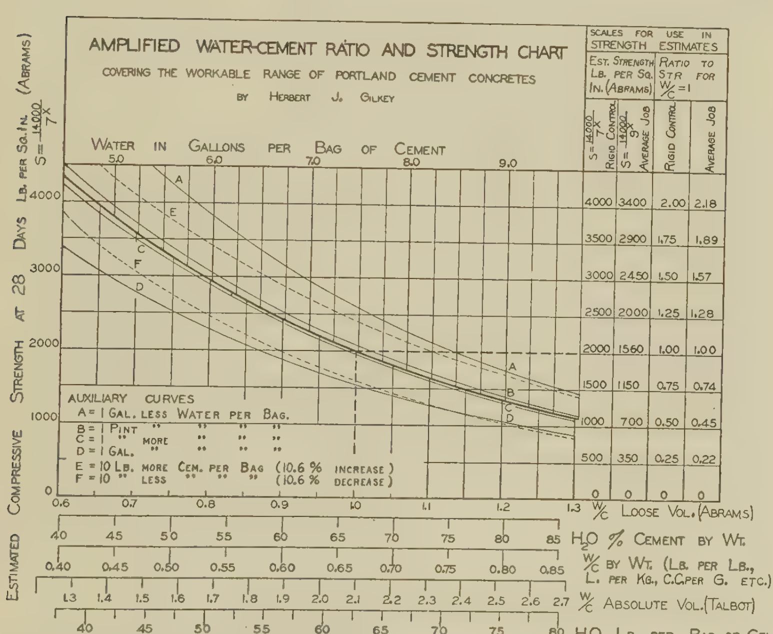

FIG. 2—WATER-CEMENT RATIO AND STRENGTH CHART FOR MAN ON THE JOB.

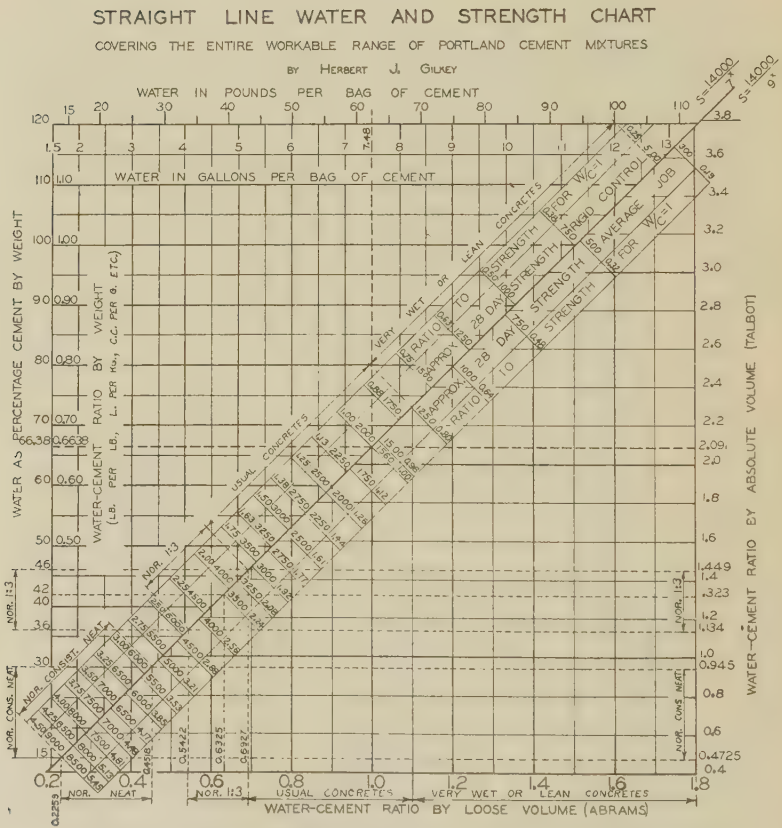

FIG. 3—STRAIGHT LINE WATER AND STRENGTH CHART IN COMPLETE FORM.

Contains practically same information as Fig. 1, but presents it by slightly different arrangement.

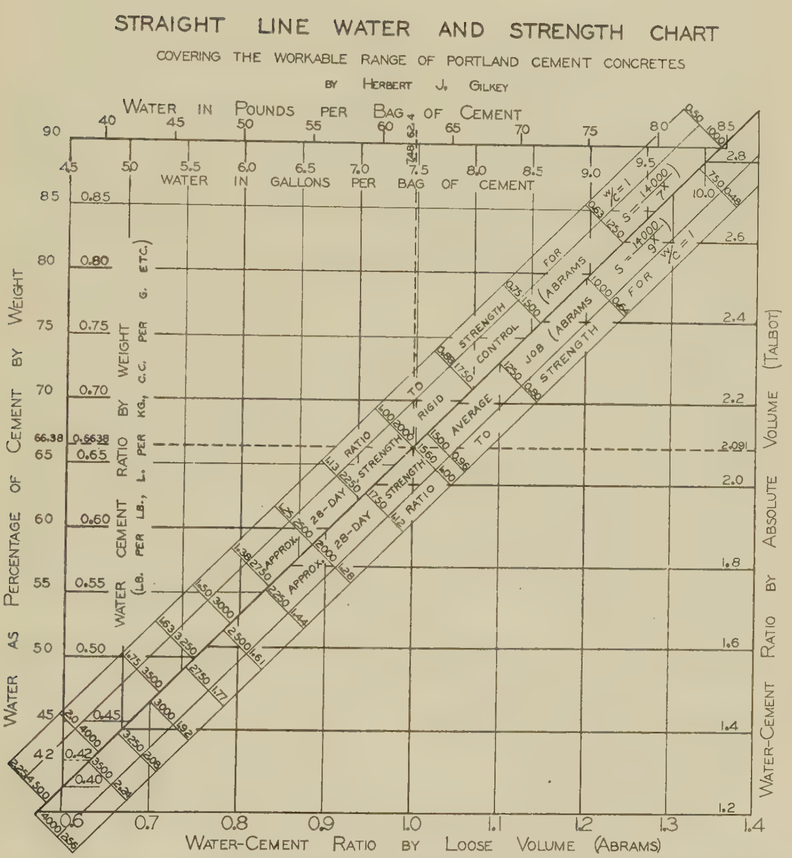

FIG. 4—STRAIGHT LINE WATER AND STRENGTH CHART COVERING JOB CONTROL VALUES.

Similar to Fig. 2 in subject matter.

High Early Strength Concrete

By Edward E. Bauer*

Concrete is a material which develops strength with age. The amount of strength acquired in any given period of time depends upon a number of factors to be discussed in this paper. In the early days concrete did not gain strength rapidly and it was found necessary to wait 28 days after making it before testing. Because of this practice of testing at 28 days, it was generally considered necessary to wait 28 days before using the concrete. During recent years tests have been made on concrete at ages earlier than 28 days so that it is possible now to use concrete structures sooner than was customary previously.

The idea of waiting 28 days before using concrete became so firmly fixed that great surprise and often alarm is expressed at the early use of concrete structures. Better materials and practices together with increased knowledge have made possible the use of concrete structures as early as 2 to 5 days after placing the concrete. Testing of specimens made from the concrete being used on any job is a good way to determine when the structure is ready to use.

It is the purpose of the writer in this paper to discuss briefly the factors which effect the rate of hardening of concrete and to present certain data bearing on this subject which he has secured during the past year.

Factors Affecting Strength—The rate at which concrete gains strength depends upon the following factors:

(1) Kind of cement
(2) Water-cement ratio
(3) Temperature
(4) Admixtures
(5) Curing
(6) Time of mixing

These factors are so interrelated that there must necessarily be some overlapping and repetition in the discussions of each.

Kind of Cement—Practically all of the cement sold in the United States is known as portland cement and is made to meet the minimum requirements of the standard specifications of the American Society for Testing Materials. Portland cement is defined by that specification to be "the product obtained by finely pulverizing clinker produced by calcining to incipient fusion an intimate and properly proportioned mixture

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of argillaceous and calcareous materials with no additions subsequent to calcination excepting water and calcined or uncalcined gypsum."[1] The cements that are made in the usual way and conform to this definition in every respect will be designated in this paper as standard portland cements. Other cements which are made in almost the same manner but which develop higher early strengths will be called special or high early-strength cements.

Standard Portland Cements—Other things remaining constant the rate at which concrete gains strength is a function of the ratio of water to cement. Concretes with the lower water-cement ratios develop strength at a more rapid rate than do those with the higher water-cement ratios. When usable early strengths are desired, those strengths may be

TABLE 1—BASIC STRENGTHS USING STANDARD PORTLAND CEMENTS[1]

Reference No.	Water-Cement Ratio, gal. per sack	Compressive Strength, lb. per sq. in. (cured wet until test)				Typical Mixes (illustrating range for a particular set of aggregates)		
		1 day	3 days	7 days	28 days	Slump, in.	Mix	Cement, bbl. per cu. yd.
1..........	7½	100	500	1100	2000	6–7	1:2:3½	1.40
2..........	6½	230	830	1530	2600	2–4	1:2:3½	1.40
3..........	6	300	1000	1800	3000	½	1:2:3½	1.40
4..........	6	300	1000	1800	3000	6–7	1:1½:3	1.65
5..........	5½	370	1230	2070	3400	2–4	1:1½:3	1.65
6..........	5	470	1500	2400	3900	½	1:1½:3	1.65
7..........	5	470	1500	2400	3900	6–7	1:1:2	2.25
8..........	4½	600	1800	2800	4300	2–4	1:1:2	2 25
9..........	4	830	2130	3170	4900	½	1:1:2	2.25

[1] From pamphlet "High Early Strength Concrete," published by the Portland Cement Association.

secured with the standard brands of portland cement by using proportions of dry materials which will permit the use of the desired water-cement ratio.

In Table 1 and Fig. 1 are given what may be termed basic time-strength relationships for water-cement ratios varying from 7½ gal. of mixing water per sack of cement to 4 gal. per sack and for ages varying from 1 to 28 days. The strengths for the various water-cement ratios at 28 days are based on the original Abrams' curve

$$S = \frac{14,000}{7^x}$$

in which S is the compressive strength at 28 days in lb. per sq. in. and x is the water-cement ratio (direct ratio: not gallons per sack of cement).[2] It should be remembered that this curve is based on laboratory tests made about 1915 and that the curing temperature was 70 deg. F. The manufacturers have made marked improvements in the quality of their

[1] A.S.T.M. Standards, 1927, Part II, p. 23.
[2] Abrams, *Design of Concrete Mixtures*, Bulletin 1, Lewis Institute.

cements in the past 15 years. The American Society for Testing Materials has raised the minimum strength requirements as rapidly as the manufacturers generally were able to meet them. Changes in the specifications were made in 1916 and 1926. Manufacturers as a rule are endeavoring to make the best cement possible and are not just trying to meet the minimum requirements of the standard specifications.

One of the methods of improving the strength-developing properties of cement has been to increase the fineness of grinding. Increased fineness of grinding has been most beneficial in producing higher early strengths. Manufacturers have also put forth special efforts to secure

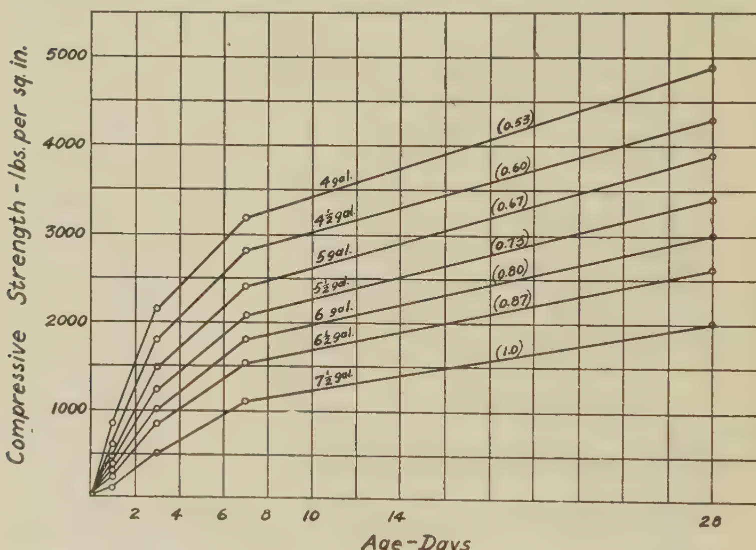

FIG. 1—BASIC RELATIONSHIPS BETWEEN AGE AND COMPRESSIVE STRENGTH FOR VARIOUS WATER-CEMENT RATIOS.
(Data shown in Table 1.)

more uniformity in the mixture of the raw materials with the result that there is not so much variation in the product of any of the plants nor from plant to plant.

When clean and structurally sound aggregates are used and when the principles of good concrete practice are followed, it is reasonable to expect that the strengths indicated in Table 1 may be secured with any of the standard brands of portland cements, except when the temperature of the concrete goes below 50 deg. F. In the summer time when air temperatures are high it is possible to secure much higher strengths than those shown. On pavement construction work in the summer the writer has secured with approximately 6 gal. of mixing water the same strength as shown in the table for 4 gal.

The curves indicate the rate at which concrete gains strength for 70 deg. F. curing. When the temperatures are higher the strength increases at a more rapid rate as will be shown later on. The probable 28-day strengths may be fairly closely estimated by means of Fig. 1 from the strengths at earlier ages.

In Table 2 and Fig. 2 are shown the results of some tests by the writer to give the relationship between age and strength for 2 of the more common mixes and water-cement ratios. For each proportion 4 different cements were used. The concrete was machine mixed, molded, cured and tested in the standard manner. The aggregates used were fairly

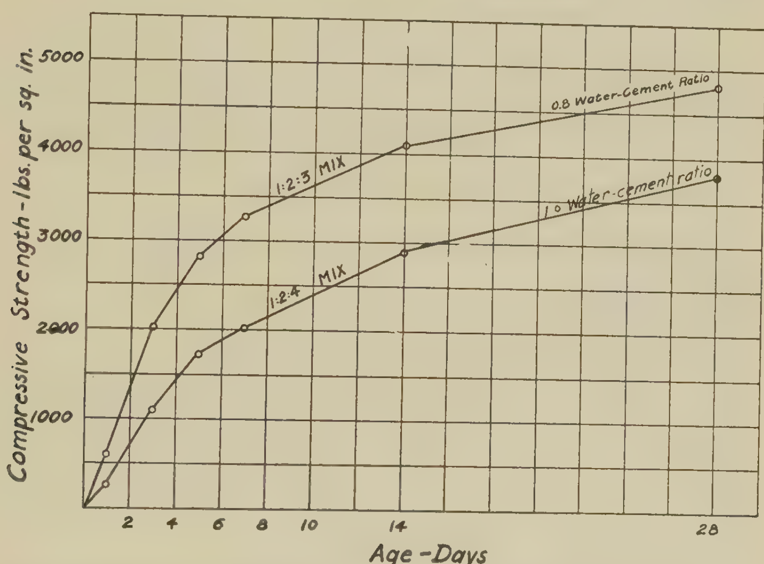

FIG. 2—RELATIONSHIP BETWEEN AGE AND COMPRESSIVE STRENGTH FOR TWO COMMON MIXES AND WATER-CEMENT RATIOS.
(Data shown in Table 2.)

coarse, which accounts for the small water-cement ratio with the 1:2:3 mix and for the large amount of slump secured for the 1:2:4 mix and the water-cement ratio of 1.

Beam Strengths—In Table 3 are given some beam strengths for the 1:2:3 mix concretes of Table 2. At the times the cylinders were made, beams were cast and cured in the same manner as the cylinders. The average curve is plotted in Fig. 3.

In Table 4 are given the results of some tests on field-made specimens, cast during the summer of 1927 with concrete being used in the city pavements at Champaign, Illinois. These beams were left on the job for 24 hr. and then placed in a moist room until tested. The unusually high 2-day value for number 5 may be due entirely to the high air

temperatures prevailing at that time and the lower 2-day value for number 6 to the lower air temperatures at that time. The average curve is shown in Fig. 3.

TABLE 2—RELATIONSHIP BETWEEN AGE AND COMPRESSIVE STRENGTH FOR STANDARD PORTLAND CEMENTS AND TWO PROPORTIONS

[LABORATORY TESTS]

VALUES IN LB. PER SQ. IN.

Age, days	Proportion, 1:2:3 Water-Cement Ratio, 0.8 Slump, 3½ in.					Proportion, 1:2:4 Water-Cement Ratio, 1.0 Slump, 9 in				
	1	2	3	4	Average	1	2	3	4	Average
1........	460	755	775	340	575	290	200	175	305	240
3........	1910	2415	1830	1850	2000	1190	975	985	1240	1100
5........	2950	2870	2770	2615	2800	1720	1665	1555	1875	1705
7........	3580	3375	3110	2985	3260	1980	1765	1980	2340	2015
14........	4280	4370	3780	3870	4075	3175	2475	2680	3155	2870
28........	5000	4610	4850	4750	4800	4100	3700	3600	3900	3825

Each value in this table is the average result for three 6 x 12-in. cylinders.

TABLE 3—RELATIONSHIP BETWEEN AGE AND BEAM STRENGTH

Standard cements
Laboratory made specimens
Mix: 1:2:3 by volume
Water-cement ratio = 0.8
Moist-room curing at 70 deg. F.

MODULII OF RUPTURE IN LB. PER SQ. IN.

Age, days	1	2	3	4	Average
1....................................	...	...	165	150	160
3....................................	475	380	425	425	425
5....................................	615	535	605	675	605
7....................................	735	585	695	815	705
14....................................	765	795	770	835	790
28....................................	810	825	950	905	870

Each value in this table is the average result of 3 breaks.

TABLE 4—RELATIONSHIP BETWEEN AGE AND BEAM STRENGTH

Standard cements
Field specimens
Mix: 1:2:3 by volume
Moist-room curing

MODULII OF RUPTURE IN LB. PER SQ. IN.

Age, days	1	2	3	4	5	6	Average
2............................	435	375	355	380	520	320	400
4............................	595	540	515	525	650	565	565
7............................	600	620	645	660	665	685	645
28............................	885	895	...	900	955	920	910

Each value in this table is the average of 2 or 3 breaks.

Special Cements—During the past few years a number of American manufacturers have been producing special cements to develop higher earlier strengths than were possible with the standard brands. Not much

is known concerning the exact method of manufacturing, except for one of them. In making this particular brand, the raw materials used in manufacturing standard portland cement are prepared and burned in the usual way. After the first grinding of the clinker it is put through the kiln again, after which it is ground in the usual manner. This gives the clinker an extra grinding and burning. The manufacturer guarantees that at least 90 per cent will pass through a 200-mesh sieve.

It should be remembered that these special cements are comparatively new and that experience with them is limited. At the present

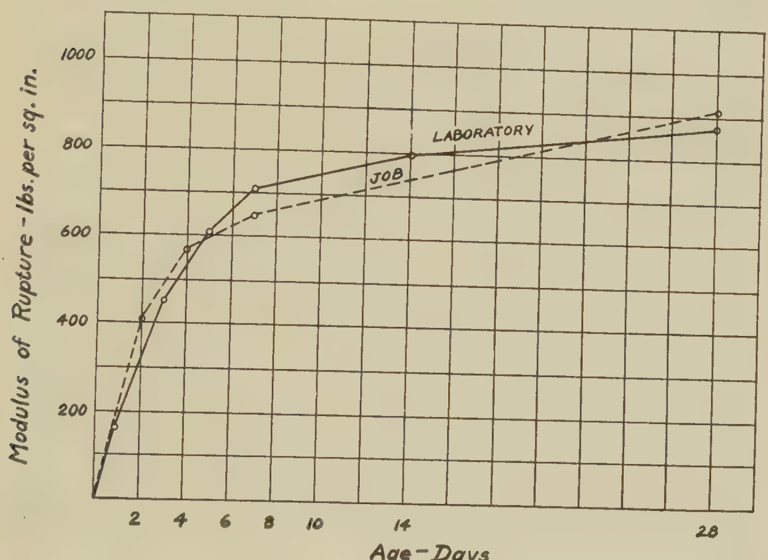

FIG. 3—RELATIONSHIP BETWEEN AGE AND BEAM STRENGTH FOR A 1:2:3 MIX AND AN 0.8 WATER-CEMENT RATIO.
(Data shown in Tables 3 and 4.)

time the manufacturers are about as familiar with their product as any one, so that it seems logical to follow rather closely their recommendations. The same rules of good practice should be followed in using these cements as would be observed with the standard cements. The writer's experience with these cements is that they do develop much higher early strengths than do the standard brands for the same water-cement ratios and conditions of making, placing and curing. The time-strength curves for these cements are different, which makes it impossible to secure average curves for this type of cement.

In Table 5 are given the results of some laboratory and field tests on concrete in which several of the special cements were used. All of the mixtures were a 1:2:3 by volume. In the laboratory tests the water-

cement ratio was held at 0.8 (6 gal. of mixing water per sack of cement). The field concrete had about the same water-cement ratio since similar materials were used in both cases. Values in the table for the field tests are for 1 cylinder each and for the laboratory tests 3 cylinders. Curves for the 1928 values are plotted in Fig. 4. A brief explanation will be given for each column in Table 5:

Column 1—Cylinders were made September 2, 1928, at Champaign, Illinois, from concrete being used in paving work. The concrete was mixed at a central plant and hauled about 1¼ miles before the cylinders were made. Cylinders were cured in a moist room at 70 deg. F. until tested.

TABLE 5—RELATIONSHIP BETWEEN AGE AND COMPRESSIVE STRENGTH

Special cements
Field and laboratory specimens
Mix: 1:2:3 by volume
Water-cement ratio = 0.8

COMPRESSIVE STRENGTHS IN LB. PER SQ. IN.

Age, days	Field Test	Laboratory Tests				Field Test Low Temperatures
		1927		1928		
	1	2	3	4	5	6
1	1770	1010	1000	2015	2145	....
2	3855	....	....	....	....	1310
3	....	3620	3400	4165	4225	2600
4	5240	....	....	....	....	3160
5	....	4475	4140	4655	5055	3930
6	5820	....	....	....	....	....
7	....	4870	4520	4895	5435	4300
13	6190	....	....	....	....	....
14	....	5220	5230	5655	6645	5000
28	[a]	5360	5510	5750	6825	5550

[a] Specimen failed to break at 7250 lb. per sq. in. At 40 days same cylinder took a load of 7670 lb. per sq. in.

Columns 2 and 3—Cylinders were made in the laboratory in the fall of 1927. Cement for columns 1 and 3 are the same. The concrete was mixed in a tilting drum mixer, molded, cured and tested in the standard way.

Columns 4 and 5—Cylinders were made in the laboratory in the fall of 1928. Cement for columns 2 and 4, and for 3 and 5 are the same. Proportions, mixing, molding, curing, and testing were same both years. Apparently during the year considerable improvement was made by the manufacturers in the ability of the cement to develop higher early strengths, the 1-day strengths in 1928 being about double those for 1927.

Column 6—Some of the same cement used for the concrete in column 4 was used by one of the Champaign contractors in building a driveway. The mix was 1:2:3 by volume. The same materials were used as for the regular paving work. The concrete was placed on November 10, 1928, while the air temperature was about 37 deg. F. Temperatures dur-

ing the first night fell a few degrees below freezing but it is doubtful whether the cylinders froze. The cylinders remained out in the open with the forms removed all the time until tested.

Cost of Special Cements—These special high early strength cement sell for about \$1.00 to \$1.50 more per barrel than do the standard brands. For a cement content of 1.5 barrels per cu. yd. of concrete, the extra cost for this cement would be between \$1.50 and \$2.25 per cu. yd. For pave-

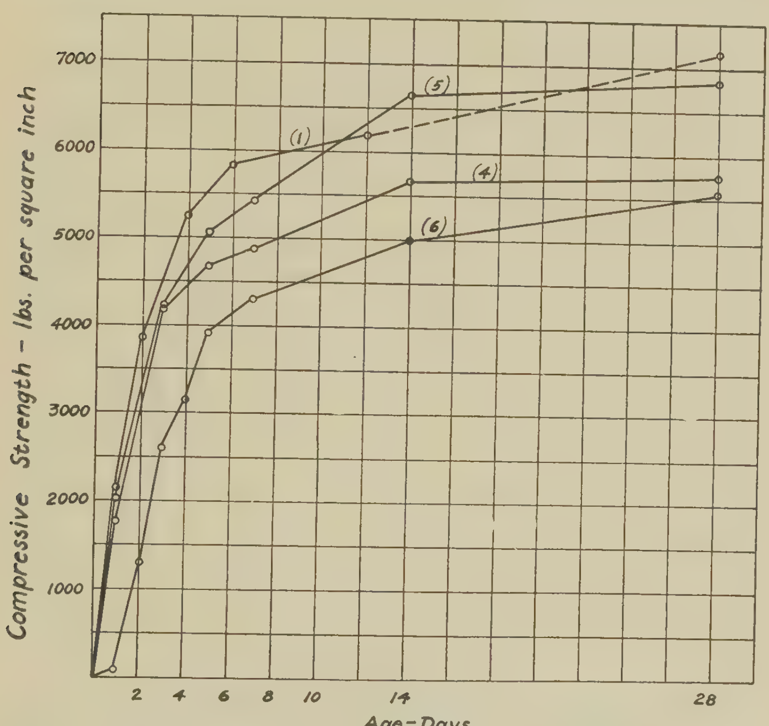

FIG. 4—RELATIONSHIP BETWEEN AGE AND COMPRESSIVE STRENGTH FOR CONCRETE WITH SPECIAL CEMENTS.
(Data shown in Table 5.)

ments 7 in. thick the extra cost for cement would be between 30 and 45c. a sq. yd. The same amount of money spent on additional standard cement will produce about the same results.

Effect of Water-Cement Ratio—As pointed out before, concretes with the lower water-cement ratios develop higher strengths than do those with the higher ratios. This holds true at all ages, so that if high or usable strengths are necessary at an early age, they may be secured through the use of concrete with the proper water-cement ratio. The

special cements come under this rule as well as the standard brands. With the special cements the water-cement ratios need not be as small as with the standard brands to secure the same strengths at the same ages.

On much of the work today the old practice still prevails of specifying the proportions of dry materials with no mention of the amount of water to be used. The wetter concretes do not gain strength as rapidly as the drier ones, nor do they gain as much. In Table 1 are given basic strengths

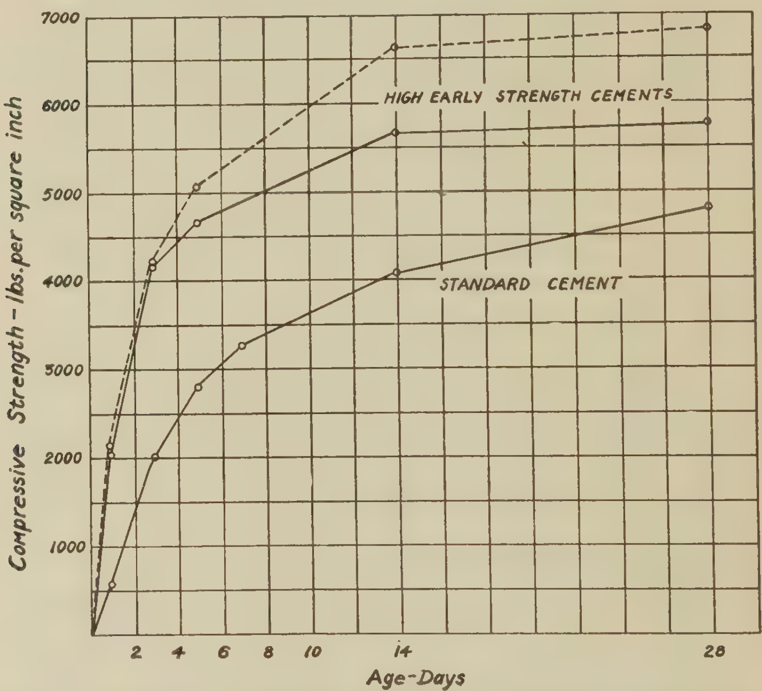

FIG. 5—RELATIONSHIP BETWEEN AGE AND COMPRESSIVE STRENGTH FOR CONCRETES MADE WITH SPECIAL AND STANDARD CEMENTS.

(Data for curves labeled *High Early Strength Cements* are shown in columns 4 and 5 in Table 5. Data for curve labeled *Standard Cement* are shown in Table 2, cement number 4 under 1:2:3 proportion.)

for 3 consistencies for each of 3 different mixes. *It should be noted that in every instance the concretes with the lower slump values (drier concrete) develop higher strengths at all ages then do the ones with the high slump values.*

In order to use the smaller water-cement ratios necessary for high early strength the amount of aggregates that can be used with each bag of cement must be reduced. The amount of the reduction will depend upon the water-cement ratio and the gradation and maximum size of the aggregate. The best proportions to use for any water-cement ratio and given materials may be most easily determined with a few trial batches.

Effect of Temperature—The rate at which a given concrete gains strength is a function of the temperature of the concrete and its age. At temperatures above freezing and below approximately 150 deg. F. higher strengths are developed for the higher temperatures. Concrete which is frozen does not gain any strength. Apparently there is a temperature between 100 and 200 deg. F. for which a maximum effect is secured. Sufficient test data are not available yet for us to know just where this critical point comes and what factors affect it. Strengths are

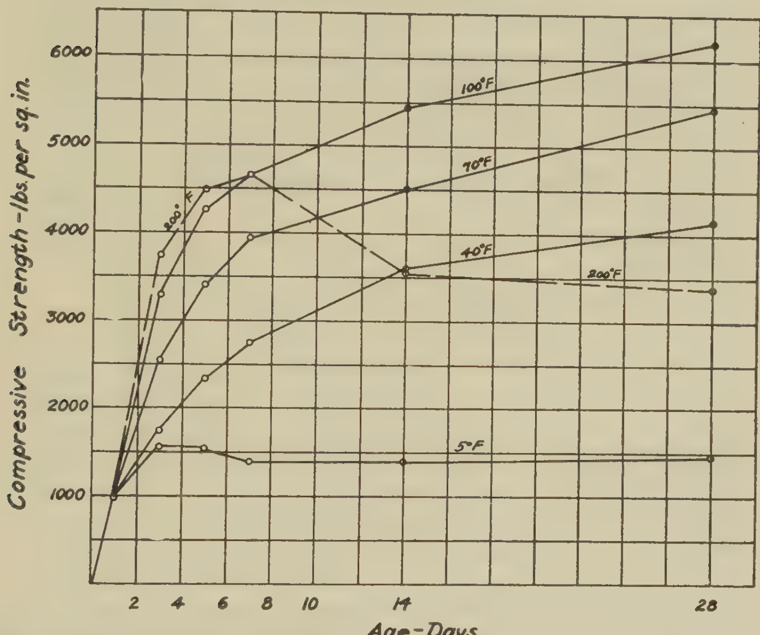

FIG. 6—RELATIONSHIP BETWEEN AGE AND COMPRESSIVE STRENGTH FOR CONCRETE CURED AT VARIOUS TEMPERATURES.
All concrete was made and cured for 1 day at a temperature of 70 deg. F. (Data shown in Table 6.)

developed at a more rapid rate for any temperature at the early ages than at the later ones.

In Table 6 and Fig. 6 are given the results of a series of tests to determine the effect of the temperature of the concrete on the strengths developed. All of the concrete was machine mixed in the laboratory at 70 deg. F. The cylinders remained in the forms for 24 hr., at the end of which time they were placed in their respective curing temperatures.

One set was placed in a refrigerator which was maintained at approximately 5 deg. F. *These cylinders gained no strength while frozen.* Cylinders were thawed out before testing.

Another set was placed in a refrigerator maintained between 32 and 40 deg. F. At no time were these cylinders frozen. Even though the temperature was just above freezing the concrete developed a fair amount of strength.

The third set was cured in the standard way, in a moist room at 70 deg. F. This is the basic or normal curve.

A fourth set was kept in water at 100 deg. F. These cylinders show a greater amount of strength at all ages.

The fifth set was placed in live steam taken directly from the university main. The temperature of the concrete was probably about 200 deg. F. Concrete materials or the concrete itself is often heated during cold weather and it was the purpose of this set to check the effect of the high temperatures on the strength of the concrete. The strengths up to 7 days were good but after that there was a serious retrogression which

TABLE 6—COMPARISON OF THE RATES OF GAINING STRENGTH OF CONCRETE CURED AT VARIOUS TEMPERATURES*

Standard portland cement
Laboratory specimens
Mix: 1:2:3 by volume
Water-cement ratio = 0.8

COMPRESSIVE STRENGTHS IN LB. PER SQ. IN.

Age, days	Curing Temperatures				
	5 deg. F.	40 deg. F.	70 deg. F.	100 deg. F.	200 deg. F.
1	....	....	1005	....	....
3	1555	1740	2530	3280	3730
5	1545	2335	3410	4260	4470
7	1395	2730	3940	4650	4670
14	1415	3600	4520	5440	3650
28	1460	4140	5440	6200	3370
49	....	....	....	....	3430

Each value in this table is the average result for 3 cylinders.

indicates that care must be exercised not to heat the concrete to high temperatures. Check tests were run to make sure no mistake had been made.

From the first set of cylinders it is quite evident that concrete does not gain strength while frozen. If the concrete had been frozen before the cement had set, the concrete would not have developed any strength. Concrete that has had an opportunity to secure some strength before freezing will continue to gain strength again once it is thawed out. Freezing at early ages does more harm than at later ages. For the richer mixes the time the concrete should be protected from freezing may be less than for the leaner ones.

In Table 7 are given the results of some field tests in which the temperature of the concrete varied. All of the concrete was made by the same contractor and was used in street paving work in Champaign or Urbana, Illinois.

* From Bauer, *Plain Concrete*, p. 173

The cylinders whose results are given in the first two columns were molded on September 1, 1928. In one case the mixer was operating on the subgrade and in the other the concrete was being hauled about 1¼ miles from a central mixing plant. Cylinders were cured in the moist room at 70 deg. F. These values are slightly higher than those in Table 2 for the laboratory made cylinders of the same proportions.

Cylinders whose results are given in column 3 were molded on November 12, 1928. The air temperature was just above freezing during that day. The cylinders were kept out in the open all the time until tested. There is a marked reduction in strength due to the lower temperatures.

A richer mix was used on one of the intersections in order to permit its use at an earlier date. Results from two series of cylinders are given in columns 4 and 5. The first set was made on October 15, 1928, when the

TABLE 7—RELATIONSHIP BETWEEN AGE AND COMPRESSIVE STRENGTH

Standard portland cements
Field specimens
Mix: variable

COMPRESSIVE STRENGTHS IN LB. PER SQ. IN.

Age	1:2:3 Mix			1:1½:2½ Mix	
	1	2	3	4	5
1 day	1025	885	100	1800	635
43½ hrs	....	....	...	2845	....
2 days	1945	1800	425	....	2080
56 hrs	....	....	...	....	2720
3 days	....	....	845	3735	....
4 days	2710	2660	1380	....:	3180
5 days	...	.. .	1800	4490	....
7 days	3960	3220	2510	4490	4030
14 days	4520	4520	3150	5650	4700
28 days	5515	5015	4170	6250	5580

Each value in this table is for 1 cylinder.

air temperatures were well above freezing. The second set was made on October 18, 1928, when the air temperatures were just above freezing. Air temperatures at night for a few nights were below freezing. Again there is an appreciable lowering of the strength due to lower temperatures. The 1:1½:2½ mix required about 1⅓ sacks more of cement per cu. yd. than did the 1:2:3 mix.

Effect of Calcium Chloride—Calcium chloride is being used now to some extent as an integral curing agent and also for the purpose of lowering the freezing point of the mixing water in cold weather. Recent tests by the Bureau of Public Roads[1] indicate that at 28 days the strength of the concrete cured with calcium chloride in the mixing water is as great as when cured with water in the usual way. If the same effect can be secured by using calcium chloride in the mixing water as by water curing, a curing problem in connection with early strength concrete will

[1] See paper by Jackson and Werner, "Field Experiments in the Curing of Pavements," in *Public Roads*, September, 1928.

have been solved. A certain amount of strength is added by moist curing during the first few days. How can concrete be cured by any one of the usual methods after it is put into service? Strengths are often sufficient at the end of 1 or 2 days to permit partial or even full use of the concrete.

Different cements react differently with calcium chloride so that it is not possible to say that any certain amount of calcium chloride will produce the maximum effect for all brands. Two per cent (generally taken as 2 lb. per sack of cement) is recommended as a safe amount to use. Greater amounts should only be used when tests indicate that a greater effect can be secured. It should not be concluded that because

TABLE 8—RELATIONSHIP BETWEEN AGE AND COMPRESSIVE STRENGTH

Standard and special cements
With and without calcium chloride admixture
Laboratory specimens
Mix: 1:2:3 by volume
Water-cement ratio = 0.8

COMPRESSIVE STRENGTHS IN LB. PER SQ. IN.

Age, days	Standard Cement						Special Cement	
	Moist Curing			Dry Curing			Dry Curing	
	Percentage of Calcium Chloride							
	0	2	4	2	3	4	0	2
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	340	1195	1185	1260	1095	920	2015	3640
3	1850	2565	2615	2530	2615	2075	4165	4950
5	2615	3065	3065	2995	3145	2345	4655	5130
7	2985	3395	3205	3715	3655	2825	4895	5385
14	3870	4170	4065	4050	4330	3470	5655	6220
28	4750	4730	4700	4590	4600	4000	5750	6550

Each value in this table is the average result for 3 cylinders.

higher strengths can be secured using calcium chloride with one special cement that the same effect can be obtained with all special cements.

In Table 8 are given the results of tests conducted by the writer in the fall of 1928 to secure time-strength relationships for various percentages of calcium chloride in a paving mixture. One standard and one high early strength cement were used.

Sets of 18 cylinders each were made with 2 and 4 per cent of calcium chloride and placed in the moist room at the end of 24 hr. Similar sets were also made with 2, 3 and 4 per cent calcium chloride and placed in the laboratory air. With 2 per cent of calcium chloride incorporated moist curing has no addition effect in developing strength. The 28-day strengths are about the same for moist curing with or without calcium chloride and for the dry curing with the admixture. The greatest gain

in strength due to calcium chloride is at one day and the effect gradually lessens until there is no effect at 28 days.

In columns 7 and 8 are given the results of tests to show the effect with a special cement. There is a very marked increase in strength at one day for this cement, 2 per cent of calcium chloride producing 3640 lb. per sq. in. as compared to 2015 without the chloride. The 28-day

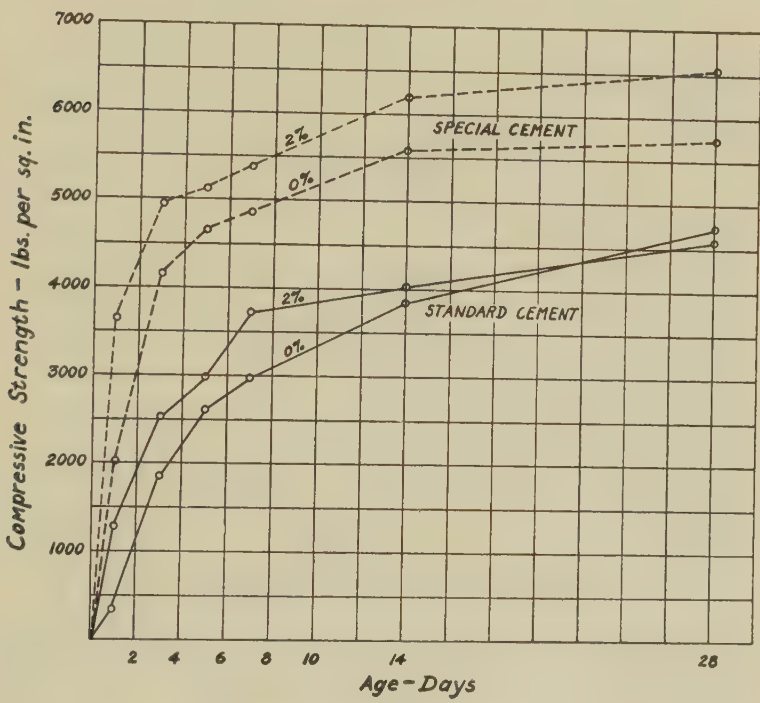

FIG. 7—RELATIONSHIP BETWEEN AGE AND COMPRESSIVE STRENGTH FOR CONCRETES MADE WITH STANDARD AND SPECIAL CEMENTS AND WITH AND WITHOUT CALCIUM CHLORIDE AS AN ADMIXTURE.

(Data shown in Table 8.)

strength with the calcium chloride is also greater than without it, but the amount is materially reduced.

Effect of Moist Curing—Unless the rate of evaporation of the mixing water out of the concrete during the first week is unusually high, moist curing has effect on the early strength of the concrete. The effect of the omission of moist curing becomes noticeable as a rule at about 28 days and becomes greater as the age of the concrete increases.

Curing should not be omitted even though the strength tests indicate good quality concrete at the early ages. In the case of the pavements, the surface layer dries out first and also receives all of the effect of traffic. Test cylinders or beams are not a measure of the quality of that surface layer unless that surface layer is kept wet, the same as the portion underneath.

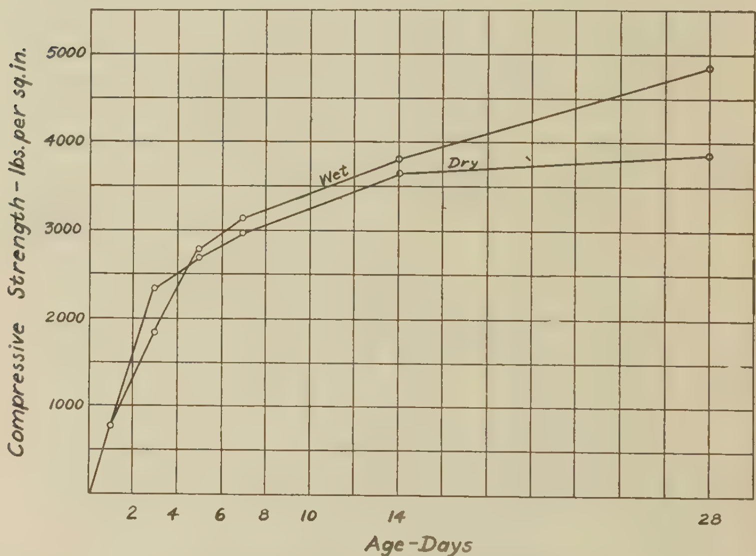

FIG. 8—RELATIONSHIP BETWEEN AGE AND COMPRESSIVE STRENGTH FOR CONCRETE CURED MOIST AND DRY.

The curves in Fig. 8 show the results for concrete which has been cured with and without moisture. The proportions were 1:2:3 by volume and the concrete was machine mixed in the laboratory.

Effect of Time of Mixing—Increasing the time of mixing is generally assumed to increase the early strength of the concrete more than its later strength. Just how much reliance in this connection can be placed on present day mixers is rather uncertain—too uncertain to be counted upon.

Discussion—High Early Strength Concrete

Mr. Morrison—About two years ago, in studying the effect of strength of cement and strength of concrete, we used three samples of cement, one of which was a standard brand of rather high early strength. We made compression cylinders, beams and tension specimens. It was interesting to observe that while the other two cements made rather normal curves, the high early strength cement decreased at twenty-eight days. That drop was visible in the beam, the compression cylinders and the tension specimens. Mr. Morrison.

There was no particular explanation for the phenomenon but about six or eight months later, in making routine tests of high early strength cement, we found that in several instances the cements were either the same strength at twenty-eight days as they were in seven days or they were slightly lower. I would like to ask Mr. Bauer if he has noticed anything similar in his work?

E. E. Bauer—I have not had any retrogression on strengths with any of the standard brands nor any of the special brands that closely resemble the standard brands. I have had one of the cements that has retrogressed in strength but there seems to be some data one way and some data the other way; some get a development of strength and others have a retrogression. Prof. Bauer.

Mr. Morrison—In fact it picked up again afterwards, didn't it? Mr. Morrison.

E. E. Bauer—No, it has never picked up. Prof. Bauer.

H. F. Gonnerman—Do you have any explanation of the falling off in strength in the case of the concrete which was heated to 200 deg. F.? Were the specimens hot at the time of the test and were they moist or dried out? Mr. Gonnerman.

E. E. Bauer—The concrete that was heated to the high temperatures was tested moist; we kept it in a steam bath and used a rather small container so that we would get a good effect from the heat. I do not know just why there should be that falling off in strength, but we checked up and found that other laboratories have found essentially the same thing—that you can heat your concrete to too high a temperature and that there will be a falling off of its strength. Prof. Bauer.

H. F. Gonnerman—Were any weights taken of the cylinders at the time of the test? Mr. Gonnerman.

E. E. Bauer—No, I have not any of that data. Prof. Bauer.

H. F. Gonnerman—The statement was made that if the extra money required to purchase high early strength cement had been expended for ordinary portland cement, approximately the same results would Mr. Gonnerman.

have been obtained, but no data are presented to show that this is true. Do you have any data showing the cost of materials?

Prof. Bauer. E. E. Bauer—Prices vary somewhat in various places as also do the gradations of the materials, so that your yield may be variable. One almost always must study the materials each time in order to determine which is going to be the cheaper.

Mr. Gonnermann. H. F. Gonnerman—I think that is true, but I was wondering if you had any data in support of this particular statement.

Prof. Bauer. E. E. Bauer—Only the series of tests I mentioned.

Mr. Ahlers. John G. Ahlers—In connection with some high speed work last fall I made a careful analysis as to the cost of portland and special cement. I found that actually we could remove forms in 2½ to 3 days and got the high early strength by using standard portland cement. Also we benefited by a lower unit cost per yard of concrete than would have been possible by using special cement.

Mr. Olsen. E. Olsen—I would like to ask the author how long he mixed that concrete?

Prof. Bauer. E. E. Bauer—The field specimens were mixed about 1½ minutes. The laboratory specimens were mixed in a small drum mixer for 2 minutes. We are using a 2-minute mix in the laboratory in all our work.

Mr. Chubb. J. H. Chubb—I want to ask a question on Table 2. Are those the same samples of cement with both the water-cement ratios?

Prof. Bauer. E. E. Bauer—I think two brands are the same. I was not able to get the same brand in all cases.

Mr. Chubb. J. H. Chubb—With your 0.8 ratio, samples 2 and 3 give you very much better strength in one and two days, then when you change the water-cement ratio to 1. How do you explain that?

Prof. Bauer. E. E. Bauer—I made no effort to line them up that way. I cannot tell you now which is which, but you should not compare them as you are doing. No. 1 in the first group might have been No. 4 in the second group.

Mr. Allyn. E. H. Allyn—Would your specimens show an increase in strength after being kept under laboratory conditions below the freezing point for 28 days and then thawed out?

Prof. Bauer. E. E. Bauer—Yes, they would. I would like to refer you to an article in *Engineering News-Record* Jan. 31, 1929, p. 179 on this subject. The longer the interval that elapsed between the time the specimen was made and the time it was frozen, the greater would be the strength that would be developed. It would seem that there is a critical age after which it does not injure concrete to freeze it. The concrete that we have frozen at one day, allowing it to remain frozen for four days and then thawed out at 70 deg. picked up in strength but it was far below the concrete which had not been frozen.

Mr. Allyn. E. H. Allyn—If the aggregate and the water are heated under outdoor conditions, how long can that heat keep the concrete warm enough that it will continue to cure?

E. E. BAUER—I am afraid that I cannot answer that. Perhaps some of the construction men have more data on that than I have. Prof. Bauer.

H. C. MCCALL—Answering the gentleman's question, in a series of tests made this winter upon a reservoir where the concrete was placed at a temperature varying from 55 to 75 deg. as it went into the forms, the temperature developed in sections about 8 ft. thick at the end of 3 days was approximately 60 deg. higher than when it went into the form. That, however, would not apply on a thin section such as a floor slab. Mr. McCall.

The Mechanism of Corrosion of Portland Cement Concrete With Special Reference to the Role of Crystal Pressure

By F. O. Anderegg*

Nearly all known materials, whether natural or artificial, are undergoing some sort of disintegrating, at least when exposed to the weather, and in accord with the laws of nature. The bacterial or fungus decay of wood, the rusting of iron, the gradual weathering of all natural rocks are examples of these great principles, which might be summed up in the terse language of the thermodynamist, "the entropy of the universe is constantly increasing."

Portland cement cannot escape these great laws, and the products made therefrom can hardly be expected to withstand where even the hardest rocks are being attacked; all that can reasonably be expected is that its products will approach the natural rocks, such as granite, in permanence. The credit for making permanent concrete products must go to the engineer, and to the contractor and his organization, as well as to the cement manufacturer. The choice of an aggregate of suitable physical properties and grading is extremely important, as is the method of application in making the concrete, while the cement, by binding the whole together, fulfills a very important function. If the binder fails, the whole concrete fails; if the improper aggregates are chosen, the best cement and best manipulation can hardly be expected to make good concrete; while the best of materials may be mishandled sadly.

Included among the natural agencies which portland cement may have occasion to withstand are: the leaching of nearly pure waters, of highly carbonated waters and of waters actually acidic; the reaction of sulfates with the aluminate compounds in the cement resulting in a volume increase; a base exchange with the lime when other salts come in contact with cement; and finally, the pressure developed by crystals growing within the concrete.

Distilled water and many natural waters, which are nearly pure, will dissolve out lime and other constituents of cement.[1] To protect concrete from this action the incorporation in the water of a little more carbon dioxide to form the insoluble carbonate has been recommended.[2] On the other hand, too much dioxide will dissolve the carbonate as bicarbonate, with harmful results.[3] The action of other stronger acids hardly needs emphasis, while the attack of sulfates is probably more general than is usually appreciated.

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ACTION OF SULFATES

When it is realized that winter rains are often appreciably acidic with sulfurous and sulfuric acids coming from burning coal to the extent of a *p*H of around 5, and to the extent of over 100 lb. of sulfuric acid per acre during one winter in a country town,[4] the importance of proper application of concrete becomes more apparent. The universal presence of sulfates in the soil means that wherever ground water has an opportunity to work its way into concrete, the insidous action of this cement "bacillus," as an early German cement worker, Michaelis,[5] called it, is constantly progressing. The proper application of the concrete, with attention to the grading and the nature of the aggregate, to the water-cement ratio, to adequate workability, and to the provision for adequate drainage, weep-holes and possibly waterproofing, will go far to reduce this disintegration to a negligible amount. Water, being the greatest agency for harm, should be rigorously excluded from the concrete.

Another method of prevention has been tried out on a small scale with some very interesting results, based upon the following line of reasoning. The mechanism of the attack by sulfates, as has been concluded by a large number of workers, is an addition of calcium sulfate to hydrated tricalcium aluminate to form a hydrated double salt, which occupies a much greater volume than the original aluminate. Then if sufficient sulfate is present to combine with all of that part of the aluminate in the cement which is hydrated before the cement has set, any expansion in volume would take place while the cement is still plastic, and without damage.

Incidentally, these experiments should have bearing on the question of specifying the SO_3 limit. With the continually finer grinding of portland cement, the amount of active material in the cement is being increased and the question arises, as to whether a specification worked out for cement ground so that 78 per cent would pass the 200 mesh sieve screen, would be fair for cement ground so that 95 per cent would pass the same sieve. In the second case, there may well be twice as much material present that will hydrate during the first hour of contact with water, as in the coarser cement. Therefore, it seems fair to raise the question as to the desirability of adding sufficient SO_3 to combine with a large part of the aluminate, in order to forestall future harmful sulfate reaction as well as to control the initial set, which is apt to be accelerated by the presence of too much unsulfated aluminate.

The first point to settle is the probable amount of the aluminate in the cement that will hydrate before the cement should stiffen to the point where expansion reactions would cause damage. Owing to the meagerness of the present knowledge as to the rate of hydration of the surface of cement clinker, any calculation based on a determination of surface area would probably have little value. Therefore, it was decided to use that part of the cement which was so fine that the probability of complete hydration of the aluminate within the given time should be large. For this purpose, the finest flour was separated from a standard portland

cement by blowing with dried air in an enlarged Pearson elutriator having an internal diameter of 9 inches. By this means, fine material was collected with diameters mostly below 0.010 mm. (10 microns) and with but few strays between 10 and 15 microns. For compression tests, one inch cubes have been found fully as satisfactory as larger specimens, provided they are molded mechanically* and have the advantage of requiring less of this material so laboriously separated.

STRENGTH DETERMINATIONS

SULFATE NOT ADDED; 1:4 MORTAR; 12.35 PER CENT WATER

Tensile Strength, lb. per sq. in.			Compressive Strength, lb. per sq. in.		
1 Day	3 Days	7 Days	1 Day	3 Days	7 Days
...	...	...	3500	3505	....
340	323	377	3090	3745	3760
379	375	360	3436	3530	3840
Average 360	349	369	3342	3593	3800

The bulk of the strength is attained within 24 hr., after which the strength changes but little. A petrographic slide made from one of the 1:4 briquettes, after breaking at 24 hr., shows a very compact structure, with the grains of sand packed closer together, as is to be expected in view of the absence of large clinker grains, which are often found separating the sand grains in a specimen made from ordinary cement.

More material was separated.

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* *Mechanical Molding of Inch Cubes for Compression Tests*

The difficulty of making 1-in. cubes for compression tests which should give concordant results, because of the size of the specimen relative to the size of the finger, has led to the development of a ram for mechanical tamping. The apparatus is simple, requiring no special parts or tools for its construction.

Two rods $\frac{3}{8}$ in., or better $\frac{1}{2}$, x 38 or 40 in. are threaded at one end and screwed into crowfoots attached to a board of about 10 x 30 in. These rods should be 6 in. apart and are spaced by two pieces of $\frac{3}{8}$ x 1-in. strap iron, one being close to the top, and the other about 6 in. from the bottom. Holes are bored in the straps so as to fit snugly over the side rods, and they should be further secured with set screws to make the frame more rigid. If the frame is strong enough, the side brace for the front rod may be eliminated, thereby appreciably improving the convenience of the apparatus; the backrod is supported by a brace made of a piece of light strap iron running diagonally from the board to about half way up the rod.

The important part of the apparatus is a removable rod of $\frac{3}{8}$-in. stock which passes through holes in the middle of the cross bars, the upper hole being $\frac{7}{16}$ in. in diameter and the lower somewhat larger. A steel plate $\frac{1}{4}$ x $\frac{3}{4}$ x $\frac{3}{4}$ in. is carefully welded to this rod 5 in. from the lower end, which is threaded for $\frac{1}{2}$ in. A piece of black pipe is selected which shall just fit nicely over the rod and cut $4\frac{1}{2}$ in. long, to share the transmission of the force of the blow. The hole in the lower cross bar should be $\frac{1}{16}$ in. larger than the outside of this pipe. A piece of brass is next selected which shall just fit nicely into the mold and a hole bored in the middle, which is then tapped to screw onto the movable rod.

For producing impact, a hole is bored in the exact center $\frac{1}{16}$-in. larger than the diameter of the center rod of a cylinder of suitable size to weigh 0.40 lb. (181.4 gr.).

In making inch cubes, the mold is filled and given 5 taps with the weight falling 30 in. The mold is than turned after finishing the first side with the trowel, and the weight is allowed. to fall 7 more times from the same height.

If glass plates are used under the molds to prevent breaking, they should rest on a rubber pad. It might be stated that the mortar is mixed in the usual way before placing in the molds. The molding is slightly faster than by hand, an aid being obtained by placing a rubber stopper on the movable rod so that the distance dropped should be correct.

The distance and number of drops were determined empirically by working with a cement on which a large number of compression tests had been run. The procedure was adjusted to reproduce the average value. With this method mean deviations are usually below 5 per cent. The method can be used to advantage for molding cylinders and larger cubes.

SULFATE NOT ADDED; 1:3 MORTAR; 14.1 PER CENT WATER

Compressive Strength, lb. per sq. in.			
1 Day	3 Days	7 Days	2 Months
4550	3800	4700	....
4550	4150	4880	6405
4500	4200	4710	6600
Average 4533	4050	4763	6503

A definite retrogression in strength was observed at 3 days but the cement had recovered by the end of the first week, and had in 2 months gained appreciably.

This separated material contained close to 4.5 per cent SO_3, as compared with an upper limit of 2 per cent in the regular cement. This, of course, was as it should be, for the soft gypsum probably aided by the high temperature of grinding was mostly broken down into a very fine powder which was blown over with the flour. The flour contained 7.2 per cent Al_2O_3 and would require 5.70 per cent SO_3 for complete combination. The addition of 2.0 per cent SO_3 as plaster of Paris was more than sufficient to combine with all of the aluminate.

SULFATE ADDED; 2.0 PER CENT SO_3 (PLASTER OF PARIS); 1:3 MORTAR; 14.8 WATER

Compressive Strength, lb. per sq. in.				
1 Day	3 Days	7 Days	28 Days	3 Months
5600	....	....	....	....
5410	4870	3980	....	....
5385	4550	4100	4800	8930
Average 5463	4710	4040	4800	8930

A marked retrogression in strength took place during the first 6 days of water storage, but at 28 days the specimens were starting to come back and by 3 months one specimen, which was stored for the second and third month in just enough carbon dioxide-free water to cover it in small sealed container, had reached a value like that of Lumnite. In fact, the behavior of this cement is somewhat similar to that of the aluminous cement, although showing more retrogression than the latter.

An examination of the specimen broken after 3 months storage failed to find more than a very slight trace of gypsum crystals. It should be pointed out that these specimens have been stored continuously in water and that other results might have been secured, if the specimens had been allowed to dry out at intervals. These experiments are being continued.

BASE EXCHANGE

The metallic part of sulfates and other salts on coming in contact with the lime of cement, enters into a base exchange, somewhat similar to that occurring in the zeolitic water softening process, resulting in an extraction of needed lime from the cement.[6] Of the various salts met

with in nature, the action of the magnesium salts is especially harmful because of acceleration given to the action by the formation and precipitation of insoluble magnesium hydroxide.[7] The best remedy is to make a compact concrete, aided probably by an effective waterproofing treatment.

Crystal Pressure From Efflorescent Action

When any material is in capillary contact on the one side with some other body containing moisture, such as the ground, and on the other side with the air, so long as there is capillary contact the moisture will flow from the ground up until it comes to the surface where it will evaporate. Those soluble salts which it has brought with it on its journey must be left behind, often having a serious disintegrating effect on the physical character of the surface. This effect is occurring every day all about us, and materials as strong and dense as granite and marble are far from being immune from it. Hence it is not surprising that concrete is also suffering from this cause.

As sea water or any other salt solution evaporates, the point is reached where the water holds all the salt it can in solution, and beyond that point the excess is thrown out of solution, almost always forming crystals which continue to grow as evaporation proceeds. This should not result in any special damage, provided the crystals have plenty of room for growth, but the trouble is that they start to form not merely on the surface but back in the pores. After a time, evaporation continues, these crystals often grow too large for their restricted habitations, and with the tremendous forces of crystal growth behind them, burst the walls which surround them. This type of disintegrating agency can most easily be prevented by adequately breaking capillary contact with the source of the moisture.

Effect of Temperature Drop

In addition to evaporation of moisture, a temperature drop aids in crystal growth and is extremely important, although it appears to have escaped attention generally. A drop in temperature by reducing the solubility of salts often results in crystal growth in the interior of the wall. If the drop is considerable and if it occurs with proper velocity, comparatively large crystals may grow within the pores of the material and each single one acting like a tiny jack, produces a cumulative force which may be great enough to disrupt the whole structure. A conclusion has been drawn that this crystal pressure is very apt to be more important in the cold weather disintegration of concrete and other building materials, than even ice pressure itself. It is supported by the following experience.

In the course of an investigation of the comparative value of certain cements, a large number of 2 x 2 x 8-in. concrete bars were made, using different mixes and different slumps. After curing for 1 week in damp

sand, the specimens were allowed to dry out for about 2 weeks. Part of the specimens were given a freezing test with a cycle of 23 hr. freezing and 1 hr. of thawing. At the end of 60 cycles, when the experiment had to be discontinued, only a few of the bars showed some slight disintegration at the corners.

Others were subjected to the regular sodium sulfate absorption test, withstanding from 25 to 75 cycles before disintegrating completely, although some 1:3 mortar bars were still only about 50 per cent gone after 75 cycles. The aggregate used in making these tests was the local river sand and gravel such as is commonly used in construction in Pittsburgh. Part of the pebbles were of soft sandstone and were readily disintegrated. Other pebbles were of smooth quartz, and these often became detached quite readily from the mortar leaving pockets on whose surfaces crystals could be seen.

Towards the end of the sodium sulfate absorption experiments, it was observed that on Monday morning after 44 hr. immersion and especially where the temperature fell over the week end, the disintegration seemed to have proceeded at a greater rate than for even two whole 24-hr. cycles. This suggested a modification for the determination of resistance to disintegration due to crystal pressure, in such a way as to reduce the opportunity for chemical action. The method was based on the idea, first, of allowing comparatively large crystals to grow within the pores, and second, that the larger the growth of the crystals the greater the disrupting action. To bring this about the concentration of the solution was lowered, and the temperature was allowed to drop slowly.

ACCELERATED METHOD OF DETERMINING RESISTANCE TO CRYSTAL PRESSURE

A 10-per cent solution of sodium sulfate was made up and placed in a pail, or other suitable container provided with a fairly tight cover. The specimens to be tested (a previous period of combined storage of 1 day in the damp closet, 6 in water and 12 in air at approximately 70 deg. F. is suggested) were placed in the solution and the whole container was surrounded with 3 or 4 layers of corrugated cardboard and placed in a refrigerator where the temperature was usually about 32 deg. F. After 44 hr. standing the crystls that had deposited in large sizes were melted by warming, and the specimens were then heated to about 235 to 240 deg. F. for 4 hr. They were then returned to the solution for another cycle.

The first test was made on bars similar to those used in the freezing test except for a longer storage in the air of 10 weeks. All the concrete bars used in this test were completely disintegrated inside of 15 cycles, indicating the effect of low temperatures in promoting disruption due to crystal pressure. Larger crystals were found in all these disintegrated specimens than were observed in the other specimens undergoing the ordinary sodium sulfate absorption test. This test is recommended for determining the resistance of concrete which is apt to come in contact

with solutions of salt, whether sea water, alkali water or just plain ground water, during periods of the year when temperatures drop to and below the freezing point.

The greater resistance offered to freezing, as compared with their comparatively poor ability to withstand the accelerated test, tends to indicate that salt crystal pressure may well be more important in the disruption of concrete and other building materials than is actual ice

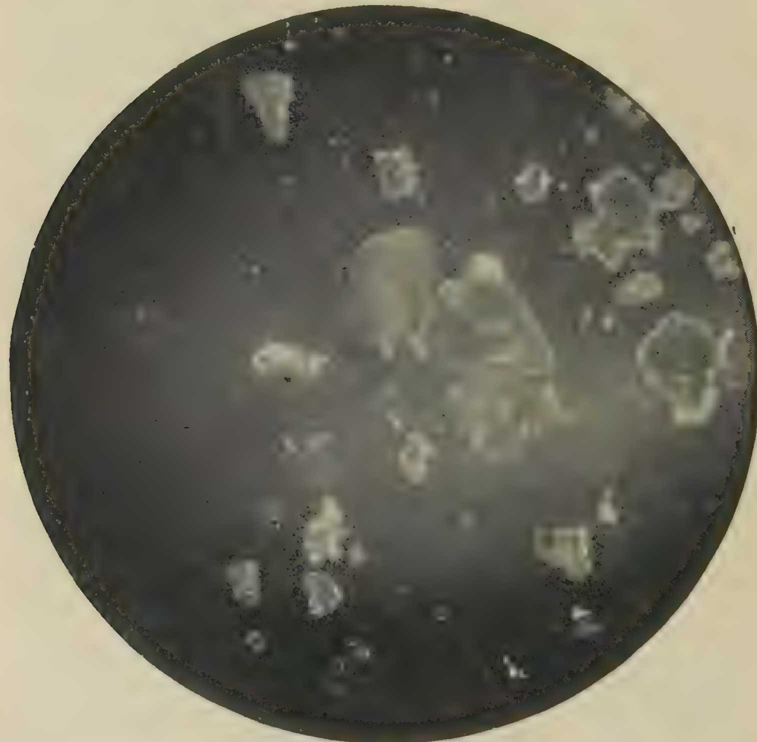

FIG. 1—CACLIUM HYDROXIDE PLATES (LIME CRYSTALS) WHICH HAVE GROWN WITHIN A PORE IN A MORTAR SPECIMEN MADE WITH CEMENT FLOUR. MAXIMUM DIMENSION, 0.60 mm. (0.024 in.). $\times$ 68.

formation. Another fact bearing on the ice damage to concrete is, that most of the water appears to be absorbed by the cement colloid and a larger part of the remainder is held within the capillaries. Now it can be demonstrated thermodynamically that the freezing point of capillary water is lower than that of free water, not only because of the increased pressure due to the surface tension,[10] but also because of the increase in concentration of dissolved salts.[11] And the same principles hold for water imbibed by a gel, but very frequently with greater intensity than

for capillary water. It is suggested, therefore, that a large part of the damage in concrete that has been ascribed to freezing of water should have been laid to salt-crystal pressure.

CRYSTALLIZING TENDENCY OF CEMENT CONSTITUENTS

The two schools of thought regarding the mechanism of the hardening of cement, the one using an explanation of crystal interlocking, the

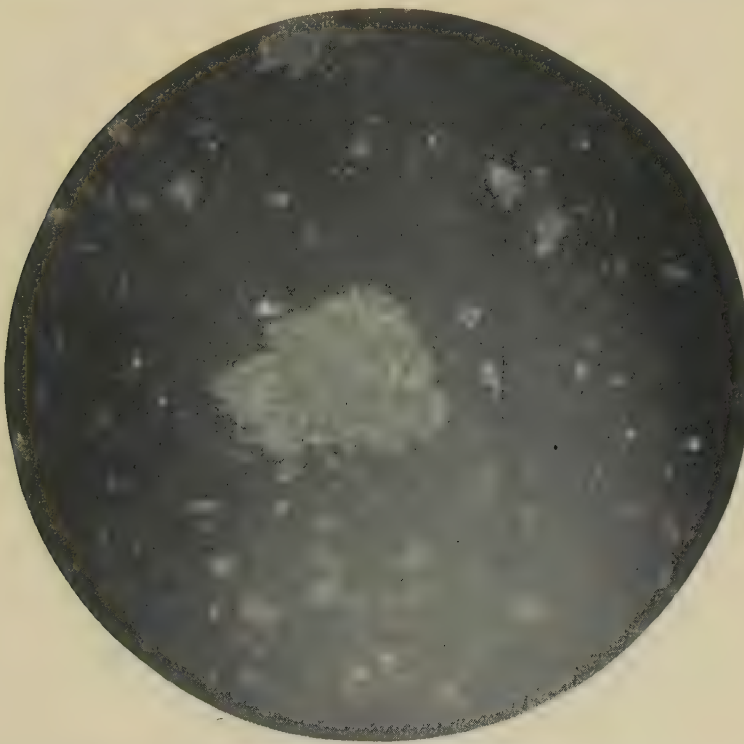

FIG. 2—HYDRATED PORTLAND CEMENT FLOUR INTERLACED WITH WHAT SEEM TO BE CRYSTALLINE NEEDLES, WHICH APPEAR AS BRIGHT LINES IN THE PHOTOGRAPH. $\times$ 68.

other ascribing the action to a silica gel, have been ably reconciled by Desch[12] and Hatschek[13] who point out the probability of the presence of both crystals and gel; even the crystals in growing are of colloidal dimension early in their life. If the crystals do grow after the concrete mass has attained its final set, it might be expected that, because of their habit of growth in certain definite directions, a weakening of structure might follow which would work against those reactions that bring about the strengthening of the concrete. Among the latter are the continued hydra-

tion of the residual cement clinker and those induration reactions which silica gels undergo with time.

This viewpoint offers an explanation of certain retrogressions in strength that have been noted in cement specimens and in concretes from time to time. The aluminous cements are known to have this characteristic and a similar effect was noted in the specimens made from cement flour, described earlier in this article. The specimen containing an excess

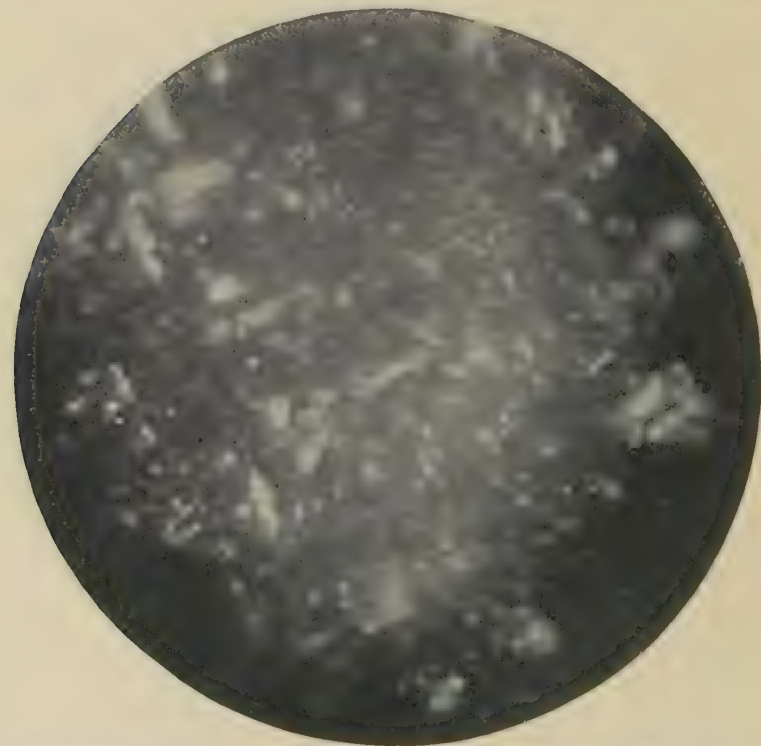

FIG. 3—AN ENLARGEMENT OF THE PHOTOGRAPH SHOWN IN FIG. 2. On gently crushing this mass the crystals are seen to be plates, smaller than, but similar to those in Fig. 1. × 250.

of SO_3 and broken at the age of 3 months was examined with great care. Calcium hydroxide crystals were found both as large plates, in the pores, some of the order of 600 microns across (Fig. 1) and as smaller crystals imbedded in a sort of haphazard manner or "brush heap" within the colloidal material. (Figs. 2 and 3.) The large plates showed interfacial angles of 120 deg., were uniaxial negative, and had indices corresponding to those assigned to calcium hydroxide.[14] The smaller crystals, imbedded in the colloidal material which obscured their outline appeared to be in

the form of needles, prisms and plates. That some of these apparent needles were in reality plates, so oriented in the clumps as to be viewed on edge, was established by disengaging them from the clump and rolling them over by applying pressure to the cover glass.

Another explanation for part of the needle-like appearances is due to an optical effect suggesting an overgrowth upon some of the calcium hydroxide crystals. These overgrowths observed on some of the large

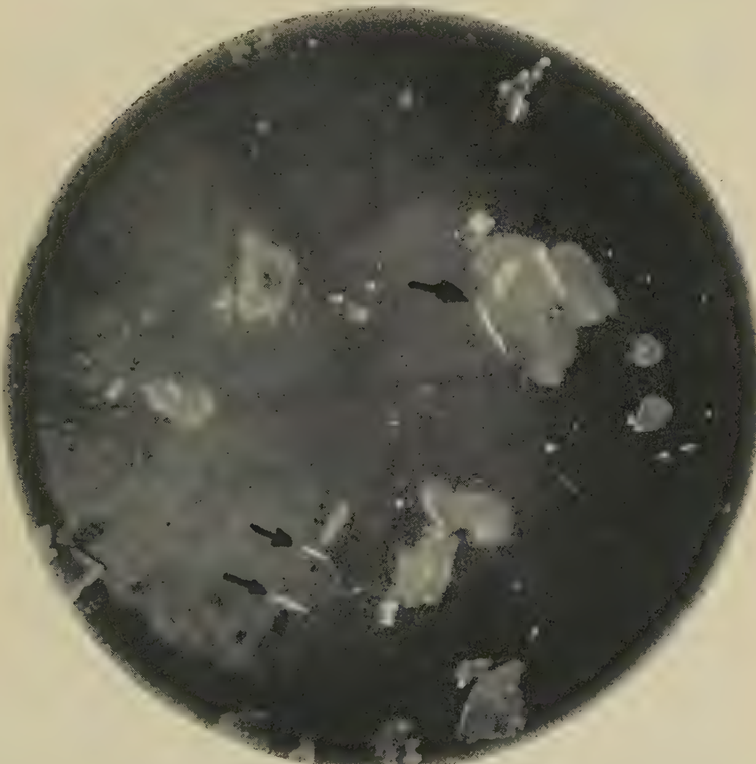

FIG. 4—CALCIUM HYDROXIDE PLATES HAVING WHAT APPEAR TO BE OVERGROWTHS AT THE EDGES.
These are much brighter than the main part of the crystal. × 100.

plates gave the plates the appearance of being pierced at the edges by needles. (Fig. 4.) The greater thickness at this point produces a greater interference color so that the needles stand out against the plate background. If these thin plates with their pale color were covered with a thin coating of amorphous material, it is quite possible that only the overgrowths would be discernable and it would appear that they were needle-like crystals penetrating the amorphous material.

Upon crushing these clumps considerable material was found which tallied in respect to indices with calcium hydroxide, but only very slight traces of material were observed which could have been gypsum. A small amount of calcite was noted, but very careful scrutiny failed to develop any evidence of sulfoaluminate crystals. The specimens made from cement flour but without the addition of more SO_3 showed, after two months storage, large calcium hydroxide crystalline growths of precisely similar character. These observations confirm the conclusion of Klein and Phillips[15] that the pressure of calcium hydroxide crystals is apt to be harmful. A conclusion that may be drawn from this experiment is that the addition of SO_3 in sufficient amount to combine with all aluminate may well be quite harmless to the concrete. This point is of real importance and should receive extensive experimental study.

Retrogression in strength may well be ascribed with Klein and Phillips, to the growth of calcium hydroxide crystals. The crystal habit results in growth chiefly on the edges of a plane so that pressure is developed outward against the confining pore walls, resulting in a diminution in strength. This should be sufficient to bring about a retrogression as noted above, or it may simply reduce the normal strength development of the concrete for a time. In the experiments with cement flour the retrogression is overcome and the strength comes back and forges ahead to high values. What probably happens is that the colloidal material gives way before the growing crystals in accordance with the known tendency of concrete to flow slightly under pressure.[16]

Acknowledgment is gratefully made to D. S. Hubbell for the very careful petrographic work and to George Walther for the compressive strength determinations.

SUMMARY

All building materials are undergoing a slow disintegration in which the presence of water is the most important factor. Portland cement concrete cannot escape the laws of nature; all that can be hoped for is to make its resistance similar to that of the most resistant natural rocks.

Concrete must withstand when rain or ground water comes in contact with it, a leaching out of lime, a reaction with the sulfate always present in natural waters (even in rain water), a crystal pressure resulting from efflorescent action which is aided both by evaporation from a surface and by drops in temperature. Therefore the importance of preventing water from entering and passing through the concrete should be emphasized.

An accelerated method of determining the resistance of concrete to crystal pressure has been developed in which the chemical attack of the sulfate is minimized, by taking advantage of crystallization behavior with slowly falling temperature.

Experiments have been made with cement flour containing from 4.5 to 6.5 per cent SO_3. Retrogressions in strength at early ages were noted, but readjustments occurred so that after 28 days and longer stor-

age remarkably high strengths were obtained. The crystals present were practically all of lime. The results fail to develop any evidence that the presence of gypsum in amounts that will react with the aluminate before the concrete had begun to set is dangerous. This has bearing on the SO_3 specification.

A method for the mechanical molding of specimens has been developed which is especially valuable for making smaller specimens, but which can also be used very advantageously for larger ones.

The conclusions of Klein and Phillips about the dangers resulting from the pressures developed by lime crystals have been confirmed, but these effects are usually only temporary because of the property of concrete of flowing to a slight extent under pressure.

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Tests of Retempered Concrete

By H. F. Gonnerman and P. M. Woodworth*

Introduction

"Time as a Factor in Making Concrete" has received much attention from the concrete industry in recent years, the American Concrete Institute devoting two entire sessions to a symposium on this subject at the 1927 Convention. The discussions at this meeting, the increasing use of central mixing plants, and the transportation of mixed concrete for long distances or for long periods before placing emphasize the need for information concerning the effect of retempering and delayed placing on the strength and workability of concrete.

The term "retempered concrete" in a strict sense is generally understood to mean concrete which has stood for some time after mixing and which is remixed with the addition of water before placing. The term is sometimes applied to concrete which is allowed to stand for some time before placing even though no additional water is added.

Most state highway specifications prohibit the use of concrete which has been mixed more than 30 min. prior to placing, and other specifications for concrete have generally prohibited the use of retempered concrete in the belief that its quality was questionable. The following paragraph from the 1924 Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete is an example:

> "34. The retempering of concrete or mortar which has partially hardened; that is, remixing with or without additional cement, aggregate, or water will not be permitted."

Some engineers have recently advocated the introduction of another "time factor" in the fabrication of concrete—premixing of the cement and water for extended periods prior to mixing with the aggregate. This premixing or so-called "prehydration" is claimed to give increased strength and other desirable properties and under certain conditions facilitates the placing of the concrete.

Although engineers have had these and related questions before them for many years only a few test data are available concerning them. This report gives the results of studies carried out for the purpose of determining the effect on the workability and the strength of the resulting concrete of (1) retempering with and without the addition of water, and

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(2) of premixing the cement and water for extended periods prior to mixing with the aggregate.

The investigation included a study of the effect of the following factors or treatments on the strength and workability of concrete of a wide range of water-cement ratios, after standing for periods up to 6 hr. in air-tight cans or in pans exposed to air of laboratory before use:

(1) Remixing without the addition of water.
(2) Remixing with addition of water to restore the concrete to its original condition of workability as measured by the flow test.
(3) Type of aggregate.
(4) Addition of aggregates to cement pastes which stood for periods up to 6 hr.
(5) Dry and soaked aggregates.
(6) Admixtures of hydrated lime or celite.
(7) Premixing cement and water for periods up to 30 min. before adding aggregate and mixing for additional periods of ½ to 10 min.

The investigation includes 8 major groups of tests on 9040 6 x 12-in. cylinders which were made from December, 1926, to March, 1927.

MATERIALS

Cement—The cement used in these tests consisted of a single shipment of 800 sacks of portland cement comprising 200 sacks each of 4 brands purchased in Chicago. About 600 of the 800 sacks were used in this series of tests.

These 4 different cements when tested in accordance with the Standard Specifications and Tests for Portland Cement of the American Society for Testing Materials (Serial Designation C 9 – 26) gave the results shown in Table 11. Lot No. 8649 represents the mixture of equal parts of the individual cements used in making the test specimens and the values in the table for its chemical analysis are the averages for the four brands; the values for physical tests are the results of tests on the composite sample.

Aggregates and Admixtures—The aggregates for mortar or concrete consisted of Elgin sand alone or combined with various coarse aggregates in the following percentages by weights:

100 per cent Elgin sand.
38.5 per cent Elgin sand and 61.5 per cent Elgin gravel.
40.2 per cent Elgin sand and 59.8 per cent Chicago limestone.
41.0 per cent Elgin sand and 59.0 per cent Wisconsin granite.
41.0 per cent Elgin sand and 59.0 per cent Chicago blast furnace slag.

Elgin sand and gravel were used in all but one of the 8 groups of tests.

In this one group Elgin sand was used in combination with each of the other 3 coarse aggregates.

The sand was graded up to the No. 4 sieve and all the coarse aggregates from the No. 4 sieve to 1½ in. The sand had a fineness modulus of 3.00, while the coarse aggregates in combination with the sand in the above percentages produced a fineness modulus of mixed aggregate in each case of 5.50.

Admixtures of commercial celite and hydrated lime purchased in the open market were used in groups 6 and 7.

Mixing Water—Water from the city of Chicago supply was used. The temperature at the time of mixing was 70 deg. in all tests. The temperature and humidity of the room during the making of the specimens showed a range as follows:

	Temperature, Deg. F.	Relative Humidity, Per Cent
Maximum	78	64
Minimum	62	15
Average	68	40

Except for the tests in Groups 1A and 2A none of the concrete was exposed to the air for more than 5 min., and it is believed that variations in humidity had little or no effect on the results.

Methods

In studying the various factors, 3 different methods of treatment of the concrete or mortar were followed:

(1) Remixing the concrete without the addition of water (Groups 1, 1A, 3, 5, and 6).
(2) Remixing the concrete with the addition of water to restore the original workability (Groups 2, 2A, and 7).
(3) Mixing a premixed cement water paste with the aggregate (Groups 4 and 8).

The detailed procedure followed for each of these methods of treatment was as follows:

(1) *Remixing without Addition of Water*—The concrete or mortar was mixed for 1 min., after all materials were in the drum, in a 3½-cu. ft. Smith Mascot mixer. The batches were sufficient in size for molding ten 6 x 12-in. cylinders. Immediately after mixing, 2 flow tests were made, and the concrete or mortar was placed in dampened, covered, air-tight metal cans for periods up to 6 hr. After standing the given period, the batch was dumped onto a dampened 4 x 10-ft. metal mixing plate and turned over once with shovels. Additional flow tests were then made and the specimens molded. Two batches were mixed for each condition on different days and two of the 10 cylinders in a batch were tested at a given age.

The various groups of tests made following this procedure were:

Group 1. 1:3, 1:4½, 1:7 concrete and 1:3 mortar mix; 5 consistencies for each mix.

Group 3. 1:4½ concrete; 5 consistencies with each of 3 coarse aggregates—granite, limestone, and slag.

Group 5. 1:3, 1:4½, and 1:7 concrete, one consistency. Aggregate soaked for 16 hr. in the water required for the batch.

Group 6. 1:4½ concrete, one consistency. Celite or hydrated lime added in amounts of 15 and 25 per cent by volume of cement.

Group 1A. 1:4½ concrete, one consistency. The procedure outlined above was followed except that the concrete was exposed to the air of the room in two shallow metal pans during the standing period. The concrete was spread out in layers, 2 to 3 in. thick, and the loss of water by evaporation was determined by weighing before and after the standing period.

(2) *Remixing with the Addition of Water to Restore the Original Workability*—The treatment in this case was the same as that given the concrete remixed without the addition of water except that additional water was added to restore the original flow. This was done after the concrete was turned over once, the batch then turned over twice with shovels prior to making the flow tests and molding the cylinders. Groups 2, 2A, and 7, made under this procedure, duplicated the mixes and consistencies in Groups 1, 1A, and 6 respectively.

(3) *Mixing Cement-Water Paste with Aggregates*—In Group 4, the cement and water were mixed for 1 min. and then placed in dampened air-tight cans for periods up to 6 hr. After the various standing periods, the aggregates were placed in the mixer, the paste added, and the batch mixed for 1 min. Following this 2 flow tests were made and the specimens immediately molded. In order to have the 0-hr. batches exactly comparable with the others, the paste was mixed as before, dumped, placed in the dampened storage can and immediately added to the aggregates in the mixer.

In Group 8, the paste was mixed for various periods up to 30 min., the aggregates were then added and the mass mixed for periods up to 10 minutes when two flow tests were made and the specimens molded.

Flow Tests—All flow tests were made in duplicate on concrete from different parts of the batch. The apparatus for making the flow test consisted of a 30-in. metal table attached to a base in such a way that it could be raised by means of a cam and dropped ½ in. by gravity. The test was made by placing a dampened truncated cone, 5 in. high, top diameter 6¾ in. and bottom diameter 10 in., on the dampened metal table and lightly puddling the concrete into the mold 25 times with a ⅝-in. round hard-rubber rod. After filling, the mold was immediately withdrawn and the table raised and dropped ½ in. 15 times in 10 seconds.

The average base diameter of the concrete after test, expressed as a percentage of the original base diameter, was the flow.

Where the word "crumbled" is used to designate flow the concrete was very dry and unworkable and did not flow but merely crumbled when jigged on the flow table.

Molding—All cylinders were molded by placing the concrete in the 6 x 12-in. metal cylinder molds in 3 layers and rodding each layer 25 times with a ⅝-in. bullet-pointed steel rod. The cylinders were covered immediately after molding with steel plates to prevent evaporation. Retempered neat cement was used to cap the cylinders 4 to 6 hr. after molding.

Curing—Cylinders were removed from the molds 16 to 20 hr. after molding, weighed, and then placed in the moist room until test. Cylinders tested at 1 day were covered with wet burlap until tested. One set of cylinders for each standing period in each group was stored in the air of the laboratory and tested at 28 days.

Testing—The moist-cured cylinders were tested damp and the air-cured cylinders were tested in a room-dry condition. The 1-day cylinders were tested in a 50,000-lb. testing machine exactly 24 hr. after the first *mixing* except those in Group 8 which were tested 24 hr. after *molding*. Compression tests at the other ages were made in a 300,000-lb. testing machine. All cylinders were loaded through a spherical bearing block placed on top of the specimens. The moving head of both testing machines traveled under load at a rate of about 0.025 in. per minute in all the tests.

DATA AND DISCUSSION OF TESTS

The principal data are reported in Tables 1 to 10. The titles of the various tables indicate the factors studied in a particular group. Some of the more important relationships from the test data are shown in Figs. 1 to 10.

In Table 1 are given the results from tests of Group 1 which include the compressive strengths at ages of 1 day to 1 yr. of the concretes and mortars, stored in air-tight cans and remixed without addition of water after periods up to 6 hr. This table also gives the flow of the concrete and mortar immediately after the first mixing and after remixing at the various standing periods, the water-cement ratios and the unit weights. In Table 2, information similar to that in Table 1 is given for the concretes and mortars of Group 2 which were remixed with the addition of water to restore the original workability.

Tables 3 and 4 contain data of tests from Groups 1A and 2A which parallel those for the 1:4½ concrete in Groups 1 and 2 except that the concrete was exposed to the air of laboratory. Table 5 gives results of tests from Group 3 on 1.4½ concrete made with 3 different coarse aggregates (granite, slag, and limestone) remixed after various periods without the addition of water; these data parallel those for the 1:4½ gravel concrete in Group 1. Results of tests from Group 4 of concrete made by

premixing the cement and water for 1 min. and allowing the paste to stand for periods up to 6 hr. prior to adding the aggregates are given in Table 6. Table 7 gives the results of tests from Group 5 of 1:4½ concrete made with aggregates soaked over night in the mixing water, the concrete being remixed after various periods without the addition of water. These data parallel those from the 1:4½ concrete in Group 1. Tables 8 and 9 give the results of tests from Groups 6 and 7 of 1:4½ concrete containing admixtures of hydrated lime or celite remixed after periods up to 6 hr. with and without addition of water. These data parallel those for the 1:4½ concrete of Groups 1 and 2.

Table 10 gives data from Group 8 showing the strengths of concrete mixed for ½ to 10 minutes made from cement water paste which had been mixed for periods up to 30 min. prior to adding the aggregate.

Basic Strength Relations for Concrete—Curves showing water-cement ratio strength relations at ages of 1 day to 1 yr. for moist-cured specimens molded immediately after mixing (0-hr. storage period) are given in Fig. 1A. These curves serve as the basis of comparison of the results obtained under the various conditions of test.

The results are very consistent at all ages; the plotted points for the 3 concretes, each mixed to 5 different consistencies (Table 1), fall close to the curves. They further confirm the important influence of the water content of the mix on the strength of concrete.

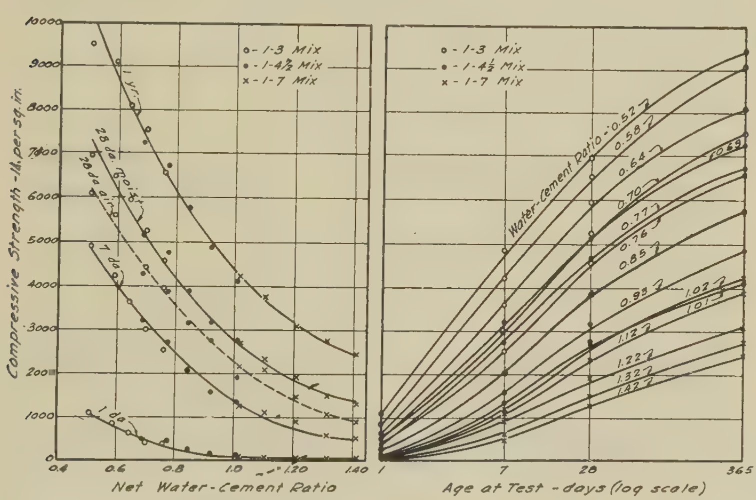


FIG. 1A (LEFT)—WATER-CEMENT RATIO STRENGTH RELATIONS FOR CONCRETE.

FIG. 1B (RIGHT)—AGE-STRENGTH RELATIONS FOR CONCRETE.

Data from Table 1.
Compression tests of 6 x 12-in. cylinders.
Aggregate sand and gravel from Elgin, Ill.
Specimens cured in moist room; tested damp.
Each value is the average of 8 or 16 tests.

The age-strength relations for different water-cement ratios in Fig. 1B are based on the same data used in plotting Fig. 1A and show the increase in strength of concrete of varying water content with age.

Strength and Flow of Concrete Remixed after Various Periods—Data from Tables 1 and 2 are plotted in Fig. 2 to show the effect on the compressive strength of 1:4½ concrete at different ages, of remixing after 2 to 6 hr. both with and without the addition of water. In general, so long

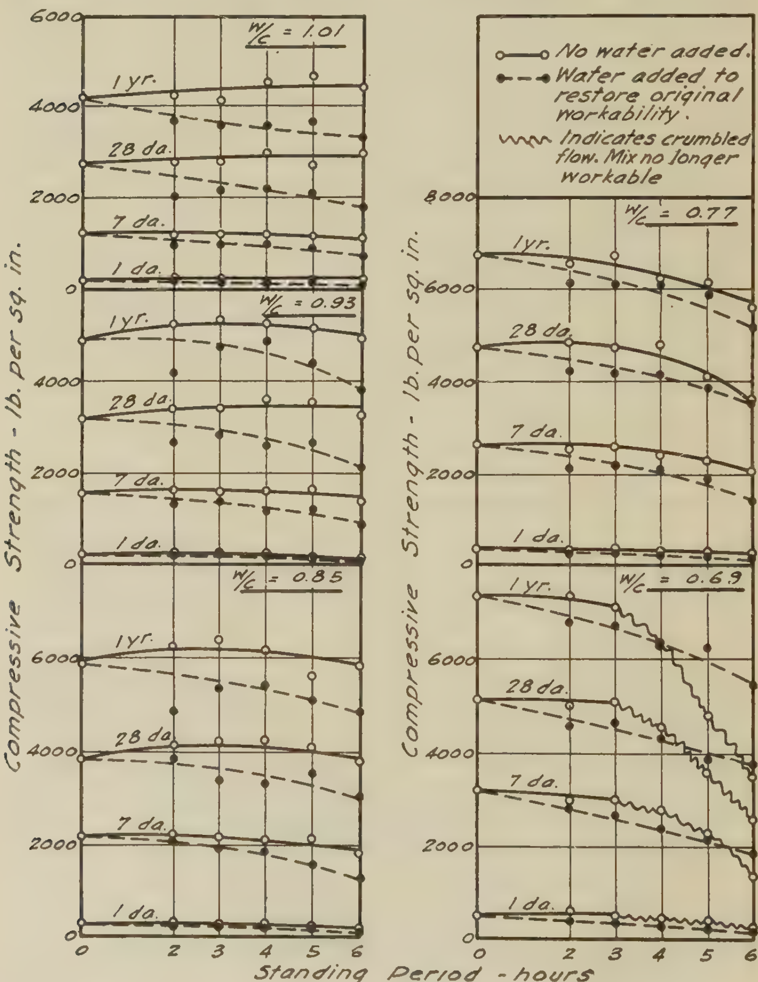


FIG. 2—EFFECT ON STRENGTH OF REMIXING WITH AND WITHOUT THE ADDITION OF WATER.
Concrete Mix 1:4½. Aggregate—Elgin sand and pebbles.
Data from Tables 1 and 2.

as the concrete remained plastic and workable the strength at any age was not materially reduced by remixing and placing without addition of water at periods up to six hours. When, however, the concrete became unworkable so that it was difficult to place in the molds without honeycombing, there was a rapid reduction in strength.

When water was added to restore the concrete to its original flow there was a reduction in strength in all cases at all ages. This reduction became more pronounced as the amount of water required to restore the concrete to its original flow increased.

Similar results were obtained with the 1:3 and 1:7 concrete mixes and the 1:3 mortar mix, but curves for these mixes are omitted in order to avoid duplication of figures.

Water-Cement-Ratio Strength Relations for Retempered Concrete—The relation between compressive strength at various ages and the net water-cement ratio of concrete restored to its original flow by the addition of water is shown in Fig. 3. By the net water-cement ratio as applied to retempered concrete is meant the total water added at the time of mixing and retempering less that absorbed by the aggregate. Since in all but Groups 1A and 2A the concrete was protected from evaporation during the standing period, no correction due to evaporation is required. The plotted values, which are taken from Table 2, are based on tests of 3 different mixtures (1:3, 1:4½ and 1:7) each mixed to 5 different consistencies and remixed with addition of water at 3 and 6 hr. Values for the other standing periods were omitted to avoid confusion. The curves which are drawn in this figure as a basis of comparison are the same as

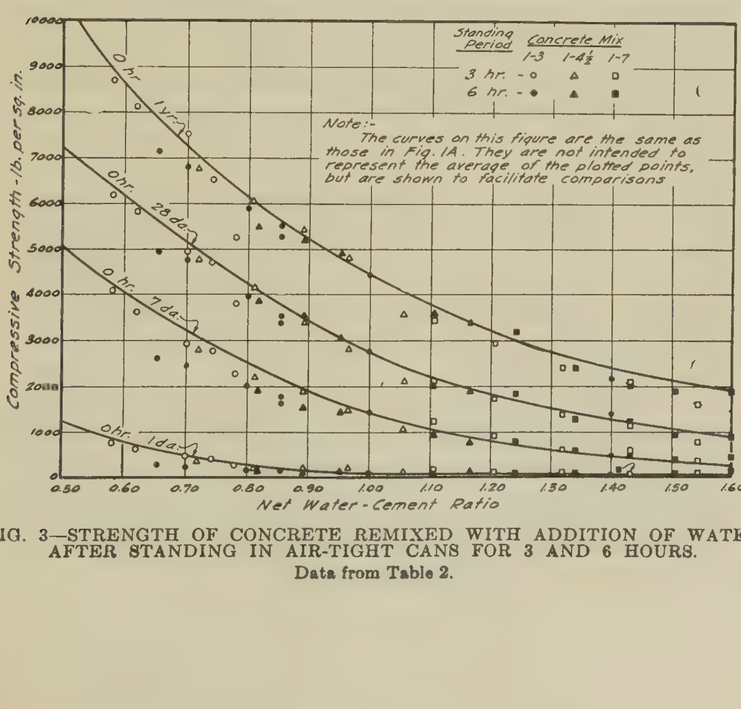


FIG. 3—STRENGTH OF CONCRETE REMIXED WITH ADDITION OF WATER AFTER STANDING IN AIR-TIGHT CANS FOR 3 AND 6 HOURS.

Data from Table 2.

those in Fig. 1A and represent the strength for the 0-hr. period. The plotted points for the 3-hr. standing periods in most cases fall close to the 0-hr. curve. This is also true for the 6-hr. standing period except for the 1:3 mix in which case the plotted points are appreciably lower than the 0-hr. curve. This indicates that 6 hr. is about the maximum standing time for satisfactory results with this mix. The strengths for the 2, 4, and 5-hr. standing periods are not plotted in this diagram, but they were close to those shown by the 0-hr. curve. These results indicate that for plastic and workable mixes and moist curing, the final net water-cement ratio is still a governing factor in the strength of concrete, even when additional water is added to restore the workability.

Workability of Concrete Remixed without Addition of Water—Data from Table 1 are plotted in Fig. 4A to show how the workability of the concrete is affected by standing for various periods when remixed without the addition of water. If a flow of about 150 be considered the lower limit of workability even the driest consistency of each of the 3 mixes was still workable after standing 2 to 3 hr. in air-tight cans.

There was a progressive reduction in the flow as the standing period increased which was more rapid for the drier consistencies. This decrease in flow was probably due to the gradual absorption of water by the dry aggregates and the accompanying stiffening of the mass.

Quantity of Water Required to Restore Workability—Fig. 4B, based on data from Table 2, shows the net water-cement ratio of the concrete when restored to its original flow after the different standing periods. As the standing time increased there was a regular increase in the amount of water required to restore the concrete to its original flow; the amount of water required after a given standing period was about the same for each of the different consistencies of a given mix.

Following are the amounts of water in gallons per sack of cement required to restore the original flow of the different mixes after the various standing periods:

Concrete Mix	Gallons per Sack, Original	Gallons per Sack Added to Restore Original Workability of Concrete after Standing for Periods Indicated				
		2 hr.	3 hr.	4 hr.	5 hr.	6 hr.
1:3	3.9 to 5.7	0.22	0.30	0.45	0.67	0.90
1:4½	5.2 to 7.6	0.22	0.30	0.40	0.67	1.05
1:4½[a]	6.4	0.30	0.45	0.67	1.05	1.40
1:7	7.6 to 10.7	0.60	0.75	0.97	1.20	1.60

[a] Concrete exposed to air of laboratory.

The 1:3 and 1:4½ mixtures held in air-tight cans required about the same amounts of water to restore the concrete to its original flow, while the 1:7 mixture required about twice as much water as the other two. With the exception noted in the table above, none of the concrete was exposed to the air for more than 5 min. and it is believed that the evaporation of the water from the concrete had no effect upon the amount of water required to restore the original flow.

Tests with Different Coarse Aggregates—The effect upon the compressive strength of remixing after standing 2 to 6 hr. without the addition of water for 1:4½ concrete from 4 different coarse aggregates is shown by the diagrams in Fig. 5. Each coarse aggregate was used in combination with Elgin sand in 5 different consistencies. The results for one consistency only are plotted in Fig. 5. The net water-cement ratios of the concrete from the four types of aggregate, which had an initial flow of about

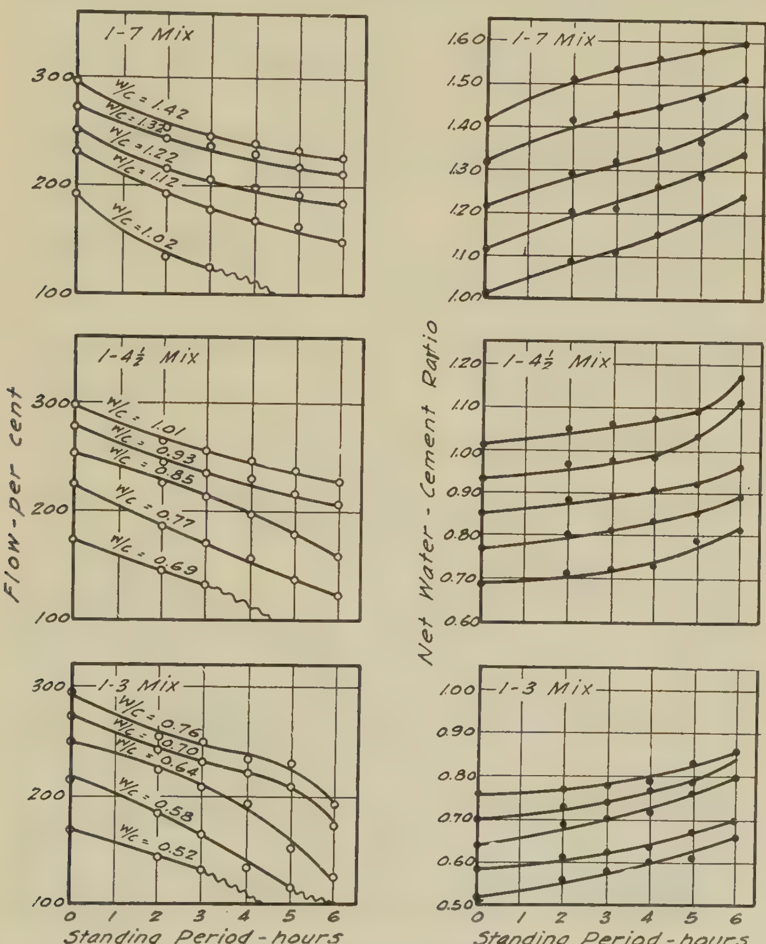


FIG. 4A (LEFT)—WORKABILITY OF CONCRETE REMIXED WITHOUT THE ADDITION OF WATER. FIG. 4B (RIGHT)—WATER-CEMENT RATIO AFTER ADDING WATER TO RESTORE WORKABILITY OF CONCRETE.

Data from Tables 1 and 2.

240 (nominal water-cement ratio 0.96) were as follows: granite, 0.91; limestone, 0.88; slag, 0.86; and gravel, 0.85. In order to facilitate comparisons of the results the average of the concrete strengths obtained from the four aggregates together with the individual strengths for each aggregate are shown in Fig. 5. These tests as well as those for the other consistencies show that there was little, if any, reduction in strength of concrete remixed at periods up to 6 hr. without the addition of water provided the concrete remained plastic and workable. The small differences in

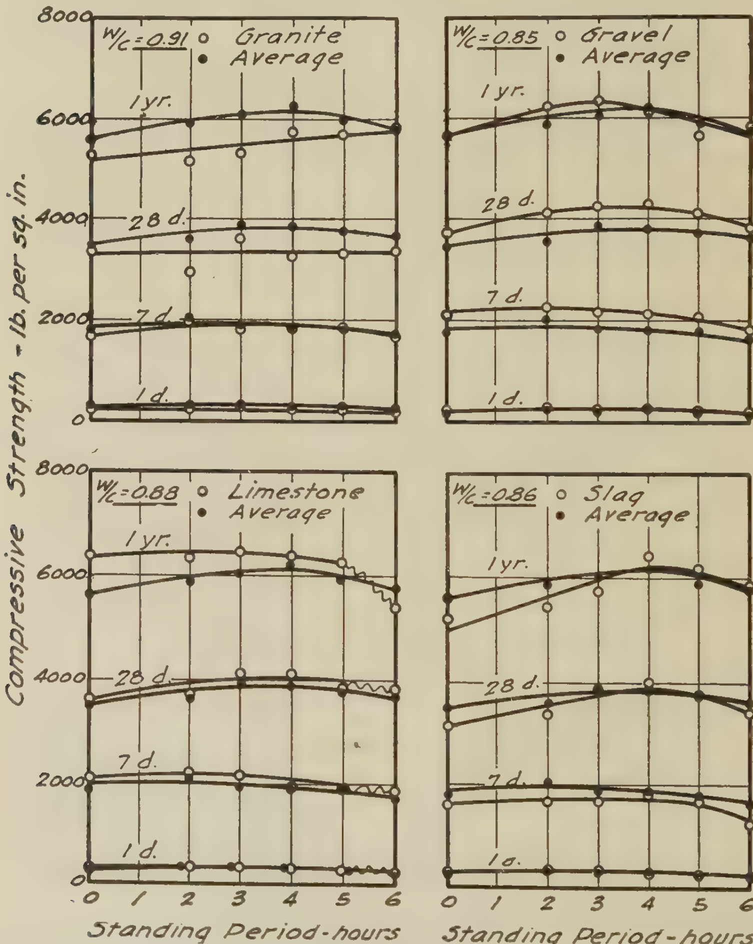


FIG. 5—STRENGTH OF CONCRETE FROM FOUR COARSE AGGREGATES REMIXED WITHOUT THE ADDITION OF WATER.
Concrete Mix 1:4½.
Data from Tables 1 and 5.

strength due to type of aggregate may be explained by differences in net water-cement ratio and by differences in adhesion between the cement paste and the coarse aggregate particles.

The workability of 1:4½ concrete from the 4 coarse aggregates is shown in Fig. 6. There was a gradual decrease in workability of the various mixtures, as measured by the flow, as the period of standing increased. For a given water-ratio, the gravel concrete showed about the same flow as the slag concrete and somewhat greater flows than the granite and limestone. In the case of the slag and limestone concrete of

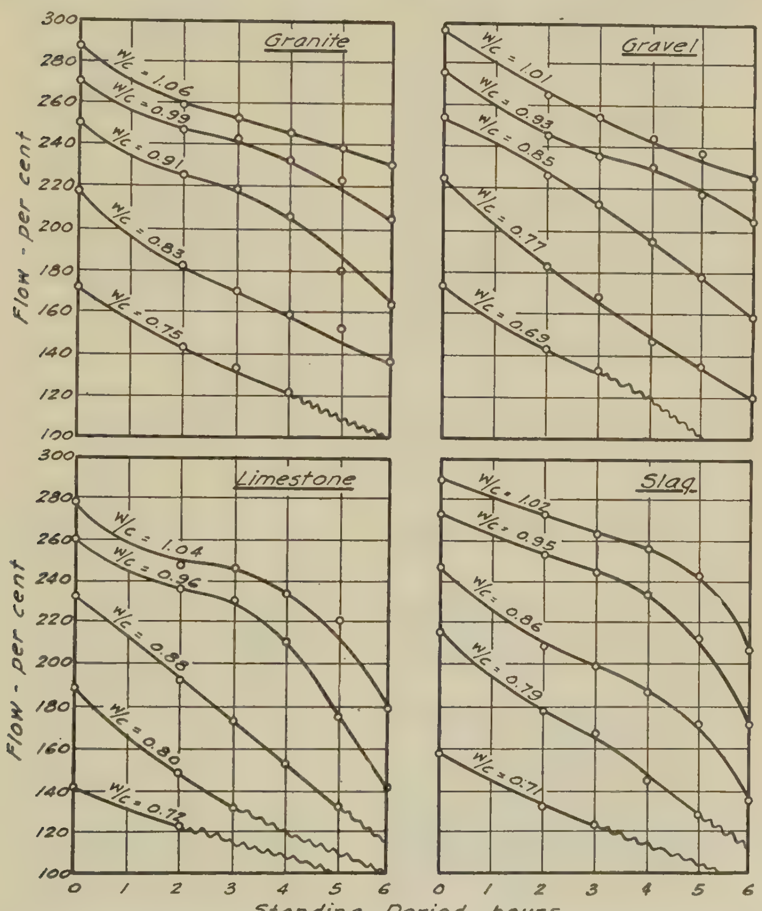


FIG. 6—WORKABILITY OF CONCRETE FROM FOUR COARSE AGGREGATES REMIXED WITHOUT THE ADDITION OF WATER.
Concrete Mix 1:4½.
Data from Tables 1 and 5.

wet consistency there was generally a marked decrease in the flow after the 3-hr. period which was probably due to the angularity of the particles of these coarse aggregates or to differences in absorption.

Strength and Workability of Concrete from Premixed Cement Paste—The diagrams in Fig. 7 compare the results of tests of concrete made with

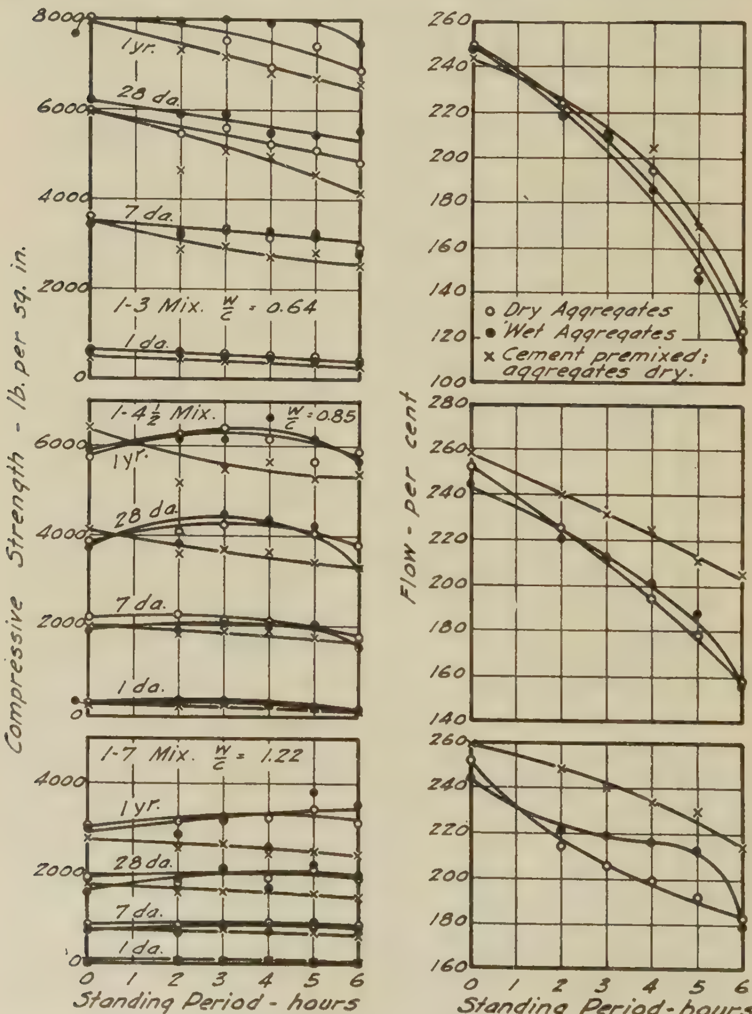


FIG. 7—EFFECT OF PREMIXING AND OF WET AGGREGATES ON STRENGTH AND WORKABILITY OF CONCRETE REMIXED WITHOUT THE ADDITION OF WATER

Data from Tables 1, 6 and 7.

cement paste premixed for periods of 2 to 6 hr. prior to adding the aggregates with tests of concrete mixed in the usual way but allowed to stand for like periods. Premixing the cement and water materially reduced the compressive strength of the 3 mixes at all ages, the reduction in strength increasing with increase in the period of standing.

The flow of the concrete made from the premixed paste which stood in air-tight cans for some time was appreciably greater than that of the concrete made by the usual method of mixing and held for the same periods except in the case of the 1:3 mix where premixing gave about the same flows as the ordinary concrete at periods up to 3 hr.

Data from Table 10 are plotted in Fig. 8 to show the effect of mixing from ½ to 10 min. on the strength of concrete made by adding the dry

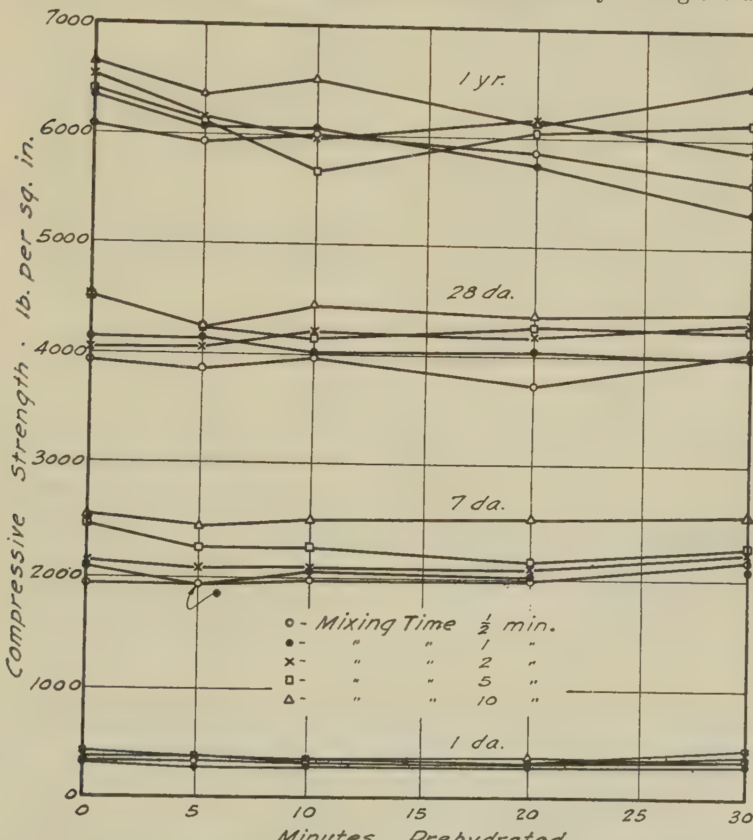


FIG. 8—EFFECT OF PREMIXING AND TIME OF MIXING ON STRENGTH OF CONCRETE.
Concrete Mix 1:4½.
Water-Cement Ratio 0.85.
Data from Table 10

aggregates to cement paste which had previously been mixed for periods up to 30 min. These curves indicate that mixing for periods up to 10 min. generally resulted in increased strength, which increase at 7 days amounted to 10 to 25 per cent of the strength of the concrete mixed for ½ min. The percentage increase in strength decreased with age at test. Premixing the cement and water for periods up to 30 min. prior to mixing with the aggregates had no beneficial effect on the strength of the concrete.

It appears from the foregoing results that adding the premixed cement-water paste to the aggregates offered some advantage over the usual method of mixing so far as increased flow after standing was concerned but caused a material reduction in strength when the paste was not used within a short time after mixing.

Comparison of Using Soaked and Dry Aggregates.—The effect of remixing after 2 to 6 hr. on strength and flow is shown by the diagrams in Fig. 7 for concrete made with both wet and dry aggregates. The wet aggregates were prepared by adding the mixing water to the dry aggregates about 16 hr. prior to mixing. The aggregates were then stored in air-tight cans until used. The quantity of water added was the same as that used in the corresponding mixes of Group 1 made from dry aggregates.

The workability of the concrete made from wet aggregates was essentially the same as that made by the usual method of mixing except in the case of the 1:7 mix where the wet aggregate showed a somewhat greater flow than the dry at periods of 2 to 5 hr. The strengths obtained with wet aggregates were about the same or slightly higher than those obtained with the dry aggregates, due probably to the reduction in the net water-cement ratio resulting from greater absorption during the long period of soaking.

Tests of Concrete Exposed to Air of Laboratory—The strength and workability of concrete exposed to the air of the laboratory for different periods immediately after mixing and before molding as compared to that of concrete placed in air-tight cans is shown in Fig. 9. Concrete exposed to air of laboratory and remixed without the addition of water generally showed an increase in strength as the time of standing increased and the strengths were generally higher than when the concrete was stored in air-tight cans. When the concrete was restored to its original workability by addition of water the strengths gradually fell off as the standing period increased and were about the same as those obtained from concrete stored in air-tight cans.

The flow of the concrete exposed to the air of the laboratory was always less than that stored in air-tight cans and the difference in flow increased with the time of standing. Evaporation of water from concrete exposed to air was the main factor causing this decrease in flow and probably accounts for the higher strengths obtained for this method of treatment. When the amount of water lost by evaporation during the standing periods is deducted from the total amount used, the amounts of water required to restore the original flow were about the same as for similar concrete stored in air-tight cans.

Effect of Admixtures in Concrete Remixed with and without the Addition of Water—Fig. 10 shows the effect on the strength and workability of a 1:4½ concrete containing a 25 per cent admixture of hydrated lime or celite by volume of cement and remixed with and without the addition of water at periods up to 6 hr. The use of the admixtures in the concrete remixed without the addition of water increased the compressive strength slightly at all ages so long as the concrete remained plastic and workable.

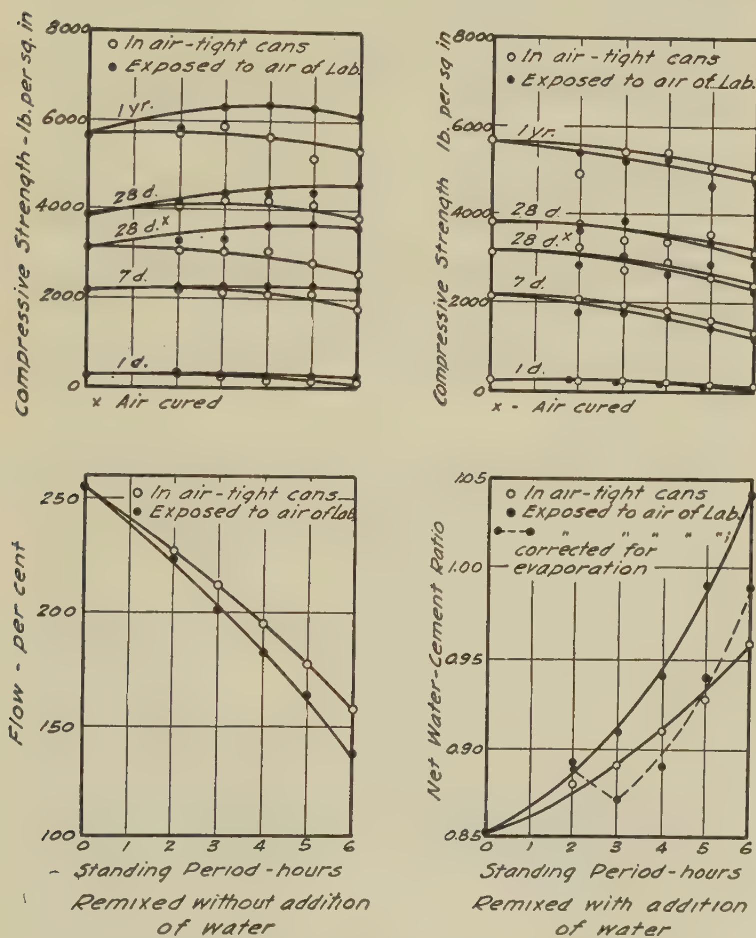


FIG. 9—COMPARISON OF CONCRETE TESTS FOR DIFFERENT EXPOSURE CONDITIONS.

Concrete Mix 1:4½. Water-Cement Ratio 0.85.
Data from Tables 1, 2, 3 and 4.

The flow, however, was considerably reduced by the addition of these admixtures. With 25 per cent of admixture by volume the flow at 4 hr. was reduced to about that of concrete without admixture at 6 hr. After the 4-hr. period the strength of the concrete containing 25 per cent of admixture fell off rapidly due to the stiffening of the concrete which made it difficult to place.

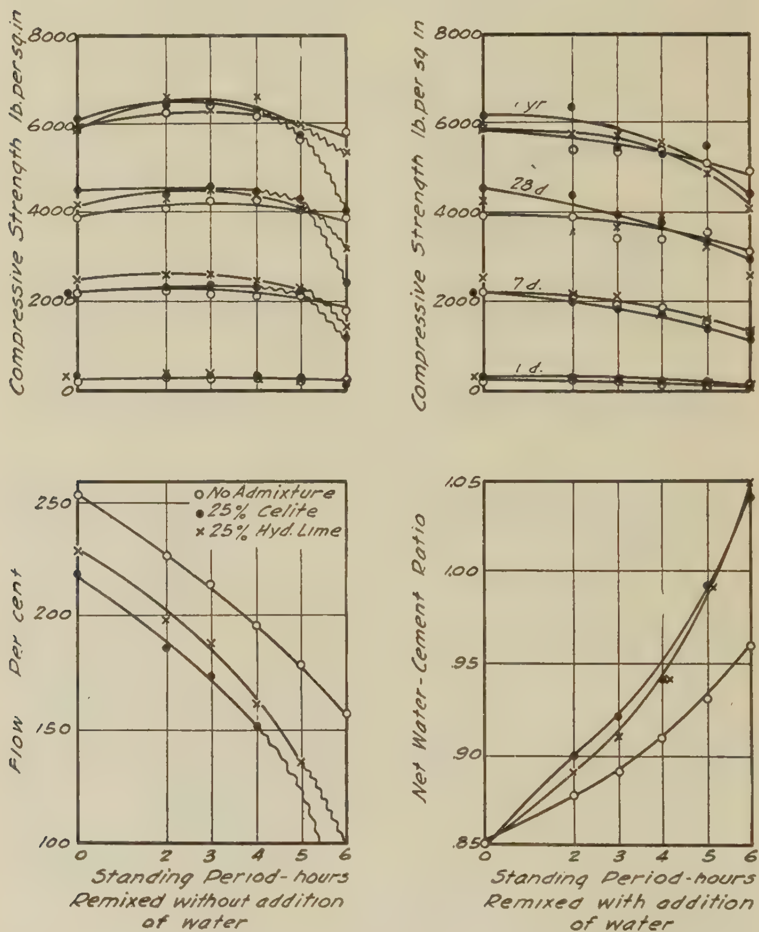


FIG. 10—EFFECT OF ADMIXTURES ON THE STRENGTH AND WORKABILITY OF CONCRETE REMIXED WITH AND WITHOUT THE ADDITION OF WATER.

Concrete Mix 1:4½. Net water-cement ratios corrected for absorption of aggregate *only*. Data from Tables 1, 8 and 9.

When the concrete containing admixtures was remixed with additional water to restore its original flow there was a falling off in strength as the time of standing increased, but the strengths were generally slightly higher than those of the concrete without admixture.

When concretes of the same mix and flow are compared, the concrete containing admixtures of hydrated lime or celite generally gave lower strength than that without admixture. The net water-cement ratios of the concretes containing admixtures, computed without attempting to evaluate the amount of water absorbed by the admixtures, were considerably greater (Fig. 10) after adding water to restore the original workability than those of the concrete without admixture.

CONCLUSIONS

The principal conclusions from the tests, which were made with a blended portland cement of normal chemical composition and physical characteristics, are as follows:

(1) The most striking result brought out by this investigation was the small loss in compressive strength due to standing for periods up to 6 hr. (protected from evaporation) of concrete remixed without the addition of water. *When the mixtures remained plastic and workable* the loss was practically nil. After the concrete ceased to be plastic the strength fell off rapidly. There was always a reduction in flow with an increase in standing period.

(2) In concrete remixed with the addition of water to restore original flow after standing for periods up to 6 hr. (protected from evaporation) the strength was reduced in accordance with the increase in the water-cement ratio resulting from the added water.

(3) When standing unprotected so that water was lost through evaporation an increase in strength resulted due to the lowered water-cement ratio.

(4) Wetting the aggregates 16 hr. in advance of mixing the concrete gave about the same strength of concrete as was obtained with dry aggregates using the same total quantity of mixing water. The use of the wetted aggregate had little, if any, effect on the flow.

(5) Premixing the cement and water for 1 minute and allowing the paste to stand for periods up to 6 hr. before adding the aggregates resulted in a gradual reduction in the compressive strength as the standing period increased, which reduction at the 6-hr. period amounted to about 20 per cent of the strength at the 0-hr. period. The flow of the concrete made from the premixed paste was generally appreciably greater than that of the concrete made by the usual method of mixing and held for like periods.

(6) Increasing the time of mixing from ½ to 10 min. resulted in an increase in compressive strength both for concrete mixed in the usual manner and for concrete made by premixing the cement and water before adding the aggregate. The percentage increase ranged from 10 to 25 per cent of the strength of concrete mixed ½ min.

(7) Tests of a 1:4½ concrete mixture containing 15 and 25 per cent of hydrated lime or celite by volume of cement, when remixed with and without the addition of water after standing for various periods, showed results similar to those obtained with concrete of the same mix without admixture. The concrete containing admixtures showed slightly higher compressive strengths than the plain but less flow after each standing period and required the addition of more water to restore the original flow. However, the plain concrete generally showed higher strength than that containing admixtures when comparisons were made on the basis of equal flow and mix.

GENERAL NOTES TO ACCOMPANY TABLES 1 TO 10

Compression tests of 6 by 12-in. concrete or mortar cylinders.

Cement: A mixture of equal parts of 4 brands of portland cement purchased in Chicago (Lot 8649).

Aggregate: Elgin sand and gravel graded 0–1½ in. (F. M. 5.50), or Elgin sand graded 0–No. 4 (F. M. = 3.00), except in Table 5 where 3 other coarse aggregates were used.

Immediately after making final flow tests, ten 6 by 12-in. cylinders were molded 2 each for test at ages indicated.

Concrete or mortar placed in 3 layers and each layer rodded 25 times with a ⅝-in. bullet-pointed steel rod.

Cylinders covered with metal plates immediately after molding and capped with retempered neat cement 4 to 6 hours after molding.

Cylinders removed from forms 16 to 20 hr. after molding and stored in moist room until test unless otherwise indicated; 1-day specimens covered with wet burlap after removal from molds until tested at 24 hr.

Unless otherwise noted each value is the average of 4 tests made in duplicate on two different days.

Table 1—Strength and Flow of Concrete and Mortar Remixed Without Addition of Water After Various Periods

(Data from Group 1)

For details of tests, see text and general notes accompanying tables.
Water-cement ratios based on—
(1) Nominal—amount of water used in first mixing.
(2) Net—nominal water corrected for absorption of aggregate.

Remixed after hr. Indicated	Water-Cement Ratio		Weight of Concrete, lb. per cu. ft.	Flow, per cent immediately after		Compressive Strength, lb. per sq. in.				
	Nominal (1)	Net (2)		Mixing	Remixing	1 Day	7 Days	28 Days	28 Days[a]	1 Year
					Concrete Mix by Volume, 1:3					
0	0.60	0.52	155	169	...	1050[b]	4820[b]	6950[b]	6060[b]	9480[b]
2	0.60	0.52	156	166	142	1070	4510	6510	5830	9410
3	0.60	0.52	156	165	131	950	4550	6330	5720	8880
4	0.60	0.52	148	167	Crumbled	600	2230	4120	3800	5100
5	0.60	0.52	138	166	Crumbled	420	1420	1940	1860	2640
6	0.60	0.52	133	169	Crumbled	150	890	1370	1140	1700[c]
0	0.66	0.58	155	216	...	840[b]	4220[b]	6510[b]	5610[b]	9090[b]
2	0.66	0.58	155	219	184	820	4100	6270	4770	8530
3	0.66	0.58	155	220	166	760	3820	5470	5230	8190
4	0.66	0.58	148	218	133	780	3830	6010	4910	8400
5	0.66	0.58	152	221	116	660	3280	5160	4180	6740[c]
6	0.66	0.58	141	221	Crumbled	270	1350	2160	1960	3590
0	0.72	0.64	154	249	...	640[b]	3630[b]	5950[b]	5610[b]	8030[b]
2	0.72	0.64	154	249	224	630	3310	5460	4920	7970
3	0.72	0.64	154	250	209	610	3320	5620	4390	7540
4	0.72	0.64	153	250	194	530	3160	5230	4070	6920
5	0.72	0.64	153	251	151	510	3180	5100	4160	7420
6	0.72	0.64	152	250	124	420	2970	4770	4010	6860
0	0.78	0.70	153	273	...	440[b]	3000[b]	5260[b]	4360[b]	7570[b]
2	0.78	0.70	153	272	242	510	3120	4940	4280	7320
3	0.78	0.70	153	273	234	460	2840	4890	4090	7190
4	0.78	0.70	153	273	221	420	2800	4980	4230	7570
5	0.78	0.70	152	273	212	380	2730	4970	4160	7540
6	0.78	0.70	151	272	174	310	2280	4650	3560	6870
0	0.84	0.76	153	296	...	360[b]	2500[b]	4580[b]	4360[b]	6600[b]
2	0.84	0.76	153	295	256	460	2630	4660	3890	6670
3	0.84	0.76	153	296	249	400	2630	4480	3890	6760
4	0.84	0.76	152	295	233	350	2440	4800	3800	6940
5	0.84	0.76	152	294	229	300	2420	4550	3690	7060
6	0.84	0.76	151	294	196	260	2160	4380	3160	6440
					Concrete Mix by Volume, 1:4½					
0	0.80	0.69	155	172	...	500[b]	3220[b]	5160[b]	4300[b]	7150[b]
2	0.80	0.69	155	172	142	520	3000	5090	4760	7440
3	0.80	0.69	155	168	131	490	3010	5100	4850	7070
4	0.80	0.69	154	175	Crumbled	450	2800	4570	3830	6360
5	0.80	0.69	151	174	Crumbled	390	2290	3550	3260	4860
6	0.80	0.69	146	172	Crumbled	240	1370	2670	2740	3500
0	0.88	0.77	154	223	...	390[b]	2680[b]	4720[b]	3900[b]	6740[b]
2	0.88	0.77	154	224	182	380	2520	4850	3850	6520
3	0.88	0.77	155	224	168	390	2580	4670	4150	6740
4	0.88	0.77	155	223	156	360	2410	4770	3930	6260
5	0.88	0.77	155	225	135	330	2390	4090	3180[c]	6170
6	0.88	0.77	153	225	120	270	2090	3690	3160	5520
0	0.96	0.85	154	252	...	280[d]	2180[d]	3870[d]	3130[d]	5720[d]
2	0.96	0.85	154	251	226	340	2230	4080[c]	3290	6230
3	0.96	0.85	154	252	212	290	2150	4230	3080	6420
4	0.96	0.85	154	252	195	280	2090	4290	3020	6120
5	0.96	0.85	153	252	178	230	2040	4030	2790	5620
6	0.96	0.85	153	251	158	190	1780	3790	2590	5830
0	1.04	0.93	153	275	...	180[b]	1580[b]	3140[b]	2760[b]	4880[b]
2	1.04	0.93	154	277	245	210	1650	3450	3080	5260
3	1.04	0.93	154	277	235	220	1620	3460	3200	5340
4	1.04	0.93	153	275	230	200	1640	3640	2780	5270
5	1.04	0.93	153	274	217	150	1650	3530	3160	5130
6	1.04	0.93	152	275	206	110	1410	3250	2510	4950
0	1.12	1.01	153	299	...	160[b]	1330[b]	2720[b]	1880[b]	4110[b]
2	1.12	1.01	154	299	265	170	1370	2790	2270	4240
3	1.12	1.01	154	299	254	160	1420	2890	2270	4140[c]
4	1.12	1.01	153	299	244	150	1380	3020	2580	4590
5	1.12	1.01	153	297	237	130	1300	2750	2430	4650
6	1.12	1.01	153	298	226	110	1240	3000	2160	4470

[a] Cured in air of laboratory. [b] Average of 8 tests. [c] Average of 3 tests. [d] Average of 16 tests.

TABLE 1—Continued

Remixed after hr. Indicated	Water-Cement Ratio		Weight of Concrete, lb. per cu. ft.	Flow, per cent immediately after		Compressive Strength, lb. per sq. in.				
	Nominal (1)	Net (2)		Mixing	Remixing	1 Day	7 Days	28 Days	28 Days[a]	1 Year
CONCRETE MIX BY VOLUME, 1:7										
0	1.20	1.02	153	193	...	150[b]	1250[b]	2690[b]	2140[b]	4280[b]
2	1.20	1.02	153	194	138	140	1190	2510	2490	4020
3	1.20	1.02	154	192	121	150	1230	2780	2160	4220
4	1.20	1.02	154	195	Crumbled	130	1180	2360	2130	3580
5	1.20	1.02	154	195	Crumbled	120	1230	2540	2340	4360
6	1.20	1.02	154	195	Crumbled	100	1150	2520	2180	3580
0	1.30	1.12	154	231	...	120[b]	1100[b]	2340[b]	2100[b]	3860[b]
2	1.30	1.12	154	230	191	110	1150	2410	2260	3760
3	1.30	1.12	154	231	179	110	1060	2210	1950	3500
4	1.30	1.12	154	230	168	100	1010	2400	1670	3940
5	1.30	1.12	154	229	161	100	1090	2460	2070	4050
6	1.30	1.12	153	230	147	70	970	2290	2170	3680
0	1.40	1.22	153	252	...	80[b]	820[b]	1910[b]	1440[b]	3080[b]
2	1.40	1.22	153	252	214	90	820	1780	1530	3150
3	1.40	1.22	153	251	206	100	900	2150	1740	3260
4	1.40	1.22	154	252	199	90	930	1920	1400	3240
5	1.40	1.22	153	251	192	70	890	2100	1560	3440
6	1.40	1.22	153	252	183	50	820	1890	1590	3140
0	1.50	1.32	154	273	...	60[b]	620[b]	1500[b]	1080[b]	2750[b]
2	1.50	1.32	154	273	243	70	640	1530	1430	2660
3	1.50	1.32	154	272	239	80	620	1450	1590	2640
4	1.50	1.32	153	274	229	70	630	1450	1460	2580
5	1.50	1.32	153	273	219	70	690	1600	1550	2920
6	1.50	1.32	154	274	214	60	660	1460	1320	3030
0	1.60	1.42	153	297	...	60[b]	540[b]	1320[b]	950[b]	2440[c]
2	1.60	1.42	154	297	252	50	470	1190	990	2440
3	1.60	1.42	154	299	246	50	460	1220	1090	2280
4	1.60	1.42	154	297	240	50	520	1340	870	2380
5	1.60	1.42	153	296	234	50	590	1330	1140	2540
6	1.60	1.42	153	298	225	50	540	1460	1240	2620[b]
MORTAR MIX BY VOLUME, 1:3										
0	0.84	0.77	142	211	...	340[b]	2330[b]	4270[b]	3670[b]	5850[b]
2	0.84	0.77	142	210	163	360	2360	4360	3960	5500[c]
3	0.84	0.77	142	214	156	300	2110	3880	3490	5370
4	0.84	0.77	142	213	148	260	2090	3900	3800	5100[c]
5	0.84	0.77	137	213	Crumbled	200	1330	2980	2230	4160
6	0.84	0.77	131	211	Crumbled	150	1220	2300	2090	3100
0	0.91	0.84	141	244	...	300[b]	2120[b]	4120[b]	3470[b]	5810[b]
2	0.91	0.84	142	243	213	290	2030	3970	3350	5680
3	0.91	0.84	142	247	201	250	2010	3710	3400	5320
4	0.91	0.84	141	247	194	240	1890	3760	3390	5400
5	0.91	0.84	141	247	178	230	1780	3670	3510	5470
6	0.91	0.84	139	244	164	170	1370	2590	2460	4180[c]
0	0.98	0.91	142	272	...	240[b]	1860[b]	3810[b]	3350[b]	5630[b]
2	0.98	0.91	142	270	242	240	1880	3850	3520	5480
3	0.98	0.91	142	272	238	210	1750	3650	3530	5030
4	0.98	0.91	141	270	231	190	1570	3300	3020	4860
5	0.98	0.91	141	268	215	180	1560	3440	2890	5160
6	0.98	0.91	140	270	193	150	1470	3040	2840	4630
0	1.05	0.98	144	300	...	210[b]	1600[b]	3580[b]	3020[b]	5180[b]
2	1.05	0.98	142	300	262	220	1540	3200	2900	4900
3	1.05	0.98	143	299	255	190	1530	2850	2830	5310
4	1.05	0.98	141	300	246	180	1440	3330	2940	5190
5	1.05	0.98	141	300	238	140	1400	3230	2620	5120
6	1.05	0.98	140	299	224	120	1270	3090	2170	4960
0	1.12	1.05	143	320	...	160[b]	1430[b]	3140[b]	2570[b]	4740[b]
2	1.12	1.05	144	320	269	210	1430	3030	3170	5430[c]
3	1.12	1.05	143	320	262	190	1380	3070	2840	4740
4	1.12	1.05	142	320	256	170	1310	2910	2730	4540
5	1.12	1.05	141	320	251	140	1290	3090	2650	5010
6	1.12	1.05	140	320	241	110	1130	2800	2340	4700

[a] Cured in air of laboratory. [b] Average of 8 tests. [c] Average of 3 tests.

Table 2—Strength of Concrete and Mortar Restored to Original Condition of Workability by Remixing with Addition of Water After Various Periods

(Data from Group 2)

Water-cement ratios based on:
(1) Nominal—amount of water used in first mixing.
(2) Final—nominal water plus that used to restore original workability
(3) Net—final water corrected for absorption of aggregate.
For details of tests see text and general notes accompanying tables.

Remixed after hr. Indicated	Water-Cement Ratio			Weight of Concrete, lb. per cu. ft.	Flow, per cent immediately after		Compressive Strength, lb. per sq. in.				
	Nominal (1)	Final (2)	Net (3)		Mixing	Adding Water	1 Day	7 Days	28 Days	28 Days[a]	1 Year
Concrete Mix by Volume, 1:3											
0	0.60	0.60	0.52	155	169	...	1050[b]	4820[b]	6950[b]	6060[b]	9480[b]
2	0.60	0.63	0.56	155	169	172	860	4440	6430	5270	9240
3	0.60	0.66	0.58	154	172	181	710	4070	6160	5320	8680
4	0.60	0.68	0.60	154	173	173	650	3920	5990	5560	8340
5	0.60	0.69	0.61	152	171	175	480	3390	5280	4580	8220
6	0.60	0.74	0.66	150	173	172	310	2560	4930	3690	7070
0	0.66	0.66	0.58	155	223	...	840[b]	4220[b]	6510[b]	5610[b]	9090[b]
2	0.66	0.69	0.61	155	222	222	690	3760	5700	4730	8540[c]
3	0.66	0.70	0.62	154	222	220	630	3620	5780	4830	8140
4	0.66	0.71	0.63	153	225	219	560	3510	5680	5190	8060
5	0.66	0.74	0.67	152	224	221	390	2980	5180	3580	7460
6	0.66	0.78	0.70	151	224	218	280	2440	4690	3640	6720
0	0.72	0.72	0.64	154	251	...	640[b]	3630[b]	5960[b]	5110[b]	8030[b]
2	0.72	0.76	0.69	153	250	250	510	3030	5170	4200	7720
3	0.72	0.77	0.70	153	252	249	460	2850	4910	4300	7510
4	0.72	0.80	0.72	153	253	250	390	2670	4690	4200	6770
5	0.72	0.85	0.77	152	250	250	270	2320	4380	3160	6520[c]
6	0.72	0.88	0.80	151	252	249	200	2000	3970	2910	5900
0	0.78	0.78	0.70	154	272	...	440[b]	3000[b]	5260[b]	4360[b]	7520[b]
2	0.78	0.81	0.73	153	273	269	430	2900	4670	4220	6900
3	0.78	0.82	0.74	153	269	271	380	2680	4670	4060	6780[a]
4	0.78	0.85	0.77	153	270	272	310	2310	4110	3300	6320
5	0.78	0.87	0.79	151	270	270	230	2110	3910	3020	5900
6	0.78	0.94	0.86	150	273	271	160	1700	3540	3010	5510[c]
0	0.84	0.84	0.76	153	300	...	360[b]	2500[b]	4580[b]	3950[b]	6600[b]
2	0.84	0.85	0.77	153	300	295	310	2260	3950	3200	5860
3	0.84	0.86	0.78	153	300	299	300	2290	3700	3300	5300[c]
4	0.84	0.87	0.79	151	300	293	260	2040	3980	3360	5580
5	0.84	0.91	0.83	151	300	297	200	1780	3730	3350	5760
6	0.84	0.94	0.86	150	300	296	150	1640	3420	3160[c]	5290
Concrete Mix by Volume, 1:4½											
0	0.80	0.80	0.69	155	190	...	500[b]	3220[b]	5160[b]	4300[b]	7150[b]
2	0.80	0.82	0.71	154	187	187	410	2860	4530	4170	6740
3	0.80	0.83	0.72	154	188	187	380	2700	4700	4030	6730
4	0.80	0.85	0.73	153	188	184	320	2450	4360	3410	6340
5	0.80	0.90	0.78	153	187	185	250	2210	3930	2960	6260
6	0.80	0.94	0.82	153	187	187	180	1910	3820	2820	5450
0	0.88	0.88	0.77	155	232	...	390[b]	2680[b]	4720[b]	3900[b]	6740[b]
2	0.88	0.92	0.80	154	229	228	300	2150	4280	3350	6090
3	0.88	0.93	0.81	154	227	227	290	2250	4140	3040	6080
4	0.88	0.94	0.83	154	228	228	240	2120	4100	3010	6080
5	0.88	0.96	0.85	153	225	228	220	1880	3910	3100	5920
6	0.88	1.00	0.89	152	228	228	150	1630[c]	3510	2590	5200
0	0.96	0.96	0.85	154	260	...	280[d]	2050[d]	3880[d]	3260[d]	5720[d]
2	0.96	0.99	0.88	154	259	255	270	2040	3840	3290	5360[c]
3	0.96	1.00	0.89	154	257	254	250	1970	3460	2750	5390
4	0.96	1.03	0.91	153	260	255	200	1830	3440	2980	5480
5	0.96	1.04	0.93	153	258	255	160	1540	3520	2610	5080
6	0.96	1.08	0.96	152	259	253	120	1370	3020	2320	4970
0	1.04	1.04	0.93	154	274	...	180[b]	1580[b]	3140[b]	2760[b]	4880[b]
2	1.04	1.07	0.96	154	273	274	180	1400	2660	2070	4190
3	1.04	1.08	0.97	154	275	275	200	1490	2810	2500	4730
4	1.04	1.10	0.98	154	279	275	150	1190	2590	2020	4930
5	1.04	1.15	1.03	153	279	273	120	1220	2680	2400	4460
6	1.04	1.23	1.11	152	278	276	90	890	2090	1690	3880
0	1.12	1.12	1.01	154	300	...	160[b]	1330[b]	2720[b]	1880[b]	4110[b]
2	1.12	1.16	1.05	154	300	300	130	1000	1990	1850	3710
3	1.12	1.17	1.06	154	300	300	130	1030	2120	1940	3590
4	1.12	1.19	1.07	153	300	300	130	1020	2170	1650	3900[c]
5	1.12	1.21	1.09	153	300	297	110	960	2090	1910	3760
6	1.12	1.29	1.17	152	299	300	80	740	1840	1400	3340

[a] Cured in air of laboratory. [b] Average of 8 tests. [c] Average of 3 tests.

TABLE 2—Continued

Remixed after hr. Indicated	Water-Cement Ratio: Nominal (1)	Water-Cement Ratio: Final (2)	Water-Cement Ratio: Net (3)	Weight of Concrete, lb. per cu. ft.	Flow, per cent immediately after: Mixing	Flow, per cent immediately after: Adding Water	Compressive Strength, lb. per sq. in.: 1 Day	7 Days	28 Days	28 Days[a]	1 Year
CONCRETE MIX BY VOLUME, 1:7											
0	1.20	1.20	1.02	153	193	...	150[b]	1250[b]	2690[b]	2140[b]	4280[b]
2	1.20	1.27	1.09	154	190	194	150	1220	2670	1970	4080
3	1.20	1.29	1.11	154	191	186	150	1280	2380	1640	3680
4	1.20	1.33	1.15	153	194	193	120	990	2160	1770	3570
5	1.20	1.37	1.19	153	190	191	90	880	2100	1520	3420
6	1.20	1.42	1.24	153	190	194	70	740	1810	1640	3140
0	1.30	1.30	1.12	153	225	...	120[b]	1100[b]	2340[b]	2100[b]	3860[b]
2	1.30	1.38	1.20	153	225	225	100	930	2080	1800	3060
3	1.30	1.39	1.21	153	226	223	110	950	1690	1430	2940
4	1.30	1.44	1.26	153	223	222	90	750	1920	1570	3140
5	1.30	1.46	1.28	153	225	224	70	700	1650	1480	2860
6	1.30	1.52	1.34	152	226	225	50	630	1290	1190	2350
0	1.40	1.40	1.22	153	251	...	80[b]	820[b]	1910[b]	1440[b]	3080[b]
2	1.40	1.47	1.29	153	252	252	70	680	1500	1130	2630
3	1.40	1.50	1.32	154	253	250	60	620	1370	1080	2360
4	1.40	1.53	1.35	153	252	251	60	540	1270	1180	2590
5	1.40	1.55	1.37	153	253	251	50	510	1360	1090	2140
6	1.40	1.61	1.43	152	253	251	40	500	1230	1080	2070
0	1.50	1.50	1.32	153	276	...	60[b]	620[b]	1500[b]	1080[b]	2750[b]
2	1.50	1.60	1.42	153	274	273	40	460	1170	910	2250
3	1.50	1.61	1.43	153	275	274	40	530	1100	670	2080
4	1.50	1.63	1.45	153	276	274	50	450	1020	900	1950
5	1.50	1.65	1.47	152	273	276	30	420	980	780	1680[c]
6	1.50	1.69	1.51	151	275	271	30	350	970	660[c]	1860
0	1.60	1.60	1.42	152	300	...	60[b]	540[b]	1320[b]	950[b]	2440[b]
2	1.60	1.70	1.52	152	300	300	40	380	850	740	1710
3	1.60	1.72	1.54	153	300	300	40	380	780	530	1600
4	1.60	1.74	1.56	153	300	298	40	400	940	550	1890
5	1.60	1.76	1.58	153	300	296	30	380	920	800	1940
6	1.60	1.78	1.60	152	300	296	40	360	970	730	1960
MORTAR MIX BY VOLUME, 1:3											
0	0.84	0.84	0.77	140	216	...	340[b]	2330[b]	4270[b]	3670[b]	5850[b]
2	0.84	0.87	0.81	142	219	215	330	2140	4140	3170	6010
3	0.84	0.90	0.83	142	217	215	290	2060	4110	3160	6040
4	0.84	0.91	0.85	142	218	213	250	1970	3700	3240	5570
5	0.84	0.94	0.88	141	218	214	200	1770	3630	3070	5800
6	0.84	0.98	0.91	140	217	214	150	1510	3280	2850	4840
0	0.91	0.91	0.84	140	248	...	300[b]	2120[b]	4120[b]	3470[b]	5810[b]
2	0.91	0.94	0.88	142	247	246	280	1910	3610	2920	5530[c]
3	0.91	0.96	0.90	142	247	246	250	1840	3650	3200	5190
4	0.91	0.98	0.91	141	249	246	230	1640	3330	2870	5050
5	0.91	1.01	0.94	141	250	247	180	1480	3360	2890	5250
6	0.91	1.04	0.97	140	250	244	120	1280	3050	2540	4790
0	0.98	0.98	0.91	141	277	...	240[b]	1860[b]	3810[b]	3350[b]	5630[b]
2	0.98	1.01	0.95	143	274	273	220	1750	3230	3020	5360
3	0.98	1.03	0.97	142	273	277	190	1570	3220	2890	5190
4	0.98	1.04	0.98	141	277	272	180	1440	3110	2760	4620
5	0.98	1.07	1.00	141	275	274	140	1370	3230	2880	5240
6	0.98	1.11	1.04	141	276	273	100	1240	2840	2800	4930
0	1.05	1.05	0.98	143	300	...	210[b]	1600[b]	3580[b]	3020[b]	5180[b]
2	1.05	1.08	1.01	144	300	300	180	1470	2990	2630	4710
3	1.05	1.09	1.02	143	300	300	160	1490	3050	2350	4760
4	1.05	1.11	1.04	142	300	300	140	1320	2840	2630	4140
5	1.05	1.13	1.06	141	300	295	110	1250	2900	2440	4840
6	1.05	1.15	1.08	141	300	300	100	1140	2750	2510	4560
0	1.12	1.12	1.05	143	320	...	160[b]	1430[b]	3140[b]	2520[b]	4740[b]
2	1.12	1.15	1.08	144	320	320	150	1270	2620	2590	4040[c]
3	1.12	1.16	1.09	144	320	320	140	1210	2510	2070	4180
4	1.12	1.17	1.10	142	320	320	120	1180	2590	2390	4130[c]
5	1.12	1.18	1.11	141	320	320	110	1170	2700	2620	4520
6	1.12	1.21	1.14	140	320	320	90	1080	2360	2240	4180

[a] Cylinders cured in air of laboratory. [b] Average of 8 tests. [c] Average of 3 tests.

TABLE 3—STRENGTH AND FLOW OF CONCRETE REMIXED WITHOUT ADDITION OF WATER AFTER EXPOSURE TO AIR FOR VARIOUS PERIODS

(Date from Group 1A)

Average temperature of room during standing period, 66 deg. F.; average relative humidity, 41 per cent. For details of tests, see text and general notes accompanying tables.

Remixed after hr. Indicated	Water-Cement Ratio		Weight of Concrete, lb. per cu. ft.	Flow, per cent immediately after		Evaporation Loss, lb.	Compressive Strength, lb. per sq. in.				
	Nominal (1)	Net (2)		Mixing	Remixing		1 Day	7 Days	28 days	28 Days[a]	1 Year
0	0.96	0.85	154	259	...	...	280[d]	2050[d]	3880[d]	3130[d]	5720[d]
2	0.96	0.85	155	257	223	0.0	330	2170	4170	3520	5880
3	0.96	0.85	155	257	201	1.3	340	2240	4430	3300	6420
4	0.96	0.85	154	259	182	1.5	290	2290	4410	3650	6470
5	0.96	0.85	154	261	164	1.6	270	2360	4460	3770	6320
6	0.96	0.85	154	260	138	1.7	240	2250	4630	3610	6160

[a] Cylinders cured in air of laboratory.
[d] Average of 16 tests.

TABLE 4—STRENGTH OF CONCRETE RESTORED TO ORIGINAL CONDITION OF WORKABILITY BY REMIXING WITH THE ADDITION OF WATER AFTER EXPOSURE TO AIR FOR VARIOUS PERIODS

(Data from Group 2A)

Average temperature of room during standing period, 66 deg. F.; average relative humidity, 41 per cent. For details of tests, see text and general notes accompanying tables.

Remixed after hr. Indicated	Water-Cement Ratio			Weight of Concrete, lb. per cu. ft.	Flow, per cent immediately after		Compressive Strength, lb. per sq. in.				
	Nominal (1)	Final (2)	Corrected (3)		Mixing	Remixing	1 Day	7 Days	28 Days	28 Days[a]	1 Year
0	0.96	0.96	0.85	155	260	...	280[d]	2050[d]	3880[d]	3130[d]	5720[d]
2	0.96	1.01	0.89	154	259	254	310	1790	3890	2850	5460
3	0.96	1.03	0.91	154	259	250	250	1790	3800	3060	5250
4	0.96	1.06	0.94	154	259	252	240	1730	3470	2650	5390
5	0.96	1.11	0.99	153	260	257	180	1480	3410	2820	4570
6	0.96	1.16	1.04	152	257	259	130	1320	2990	2250	4730

[a] Cylinders cured in air of laboratory.
[d] Average of 16 tests.

TABLE 5—STRENGTH AND FLOW OF CONCRETE FROM 3 TYPES OF COARSE AGGREGATE REMIXED WITHOUT ADDITION OF WATER AFTER VARIOUS PERIODS

(Data from Group 3)

Concrete mix: 1:4½ by volume.
For details of tests, see text and general notes accompanying tables.

Remixed after hr. Indicated	Water-Cement Ratio		Weight of Concrete, lb. per cu. ft.	Flow, per cent immediately after		Compressive Strength, lb. per sq. in.				
	Nominal (1)	Net (2)		Mixing	Remixing	1 Day	7 Days	28 Days	28 Days[a]	1 Year
Elgin Sand and Wisconsin Granite										
0	0.80	0.75	153	173	...	420	2550	4570	3870	6640
2	0.80	0.75	153	172	143	460	2670	4640	3800	7210
3	0.80	0.75	153	169	134	400	2830	4830	3530	6240
4	0.80	0.75	153	170	121	370	2650	4320	3460	6580
5	0.80	0.75	152	172	Crumbled	340	2370	4200	3060	6340
6	0.80	0.75	146	173	Crumbled	250	1640	3120	2440	4680
0	0.88	0.83	152	220	...	310	2230	3960	3170	6020
2	0.88	0.83	153	217	182	380	2580	4510	3230	6050
3	0.88	0.83	152	217	170	340	2480	4460	3220	6850
4	0.88	0.83	152	218	159	320	2280	3970	3110	6200
5	0.88	0.83	152	217	152	250	2310	4010	2870	6200[c]
6	0.88	0.83	151	219	137	220	2080	3770	2950	6080
0	0.96	0.91	152	252	...	220	1630	3330	2670	5260
2	0.96	0.91	152	249	225	240	1980	2970	2650	5170
3	0.96	0.91	152	249	219	260	1790	3660	3060	5360
4	0.96	0.91	151	249	206	250	1810	3230	2750	5740
5	0.96	0.91	151	250	180	210	1890	3340	2740	5620
6	0.96	0.91	151	251	163	170	1620	3380	2730	5860
0	1.04	0.99	151	269	...	180	1520	2690	2350	4580[c]
2	1.04	0.99	151	269	247	210	1380	2630	2120	4930
3	1.04	0.99	152	268	243	200	1510	3200	2510	5050
4	1.04	0.99	152	270	233	200	1440	3380	2410	4610
5	1.04	0.99	151	272	223	160	1440	3040	2200	5150
6	1.04	0.99	150	272	205	140	1340	3230	2530	5190
0	1.12	1.06	150	287	...	140	1100	2170	1480	3160
2	1.12	1.06	151	287	259	140	940	2110	1590	3770
3	1.12	1.06	151	287	252	150	1130	2480	1800	3340
4	1.12	1.06	151	287	245	130	1100	2230	1740	4250
5	1.12	1.06	150	288	238	120	1140	2660	2310	4000
6	1.12	1.06	150	286	230	100	1050	2780[c]	2260	4220
Elgin Sand and Chicago Limestone										
0	0.80	0.72	155	144	...	560	3240	5210	4130	8080
2	0.80	0.72	155	141	122	550	3240	5270	4490	7510
3	0.80	0.72	155	141	Crumbled	560	3000	5070	4250	7180
4	0.80	0.72	150	142	Crumbled	470	2190	4090	3380	4560[c]
5	0.80	0.72	144	142	Crumbled	280	1460	2860	2470	3410
6	0.80	0.72	139	143	Crumbled	180	1190	1610	2160	2560[b]
0	0.88	0.80	155	187	...	450	2790	4900	3620	7210
2	0.88	0.80	155	189	149	470	2680	4440	3810	7260
3	0.88	0.80	155	188	131	440	2590	4520	3820	7280
4	0.88	0.80	154	189	Crumbled	380	2520	4750	3780	7210
5	0.88	0.80	152	191	Crumbled	330	2320	4020	3060	6100
6	0.88	0.80	148	189	Crumbled	250	1790	2870	2700	4200

[a] Cylinders cured in air of laboratory.
[b] Average of 2 tests.
[c] Average of 3 tests.

TABLE 5—Continued

Remixed after hr. Indicated	Water-Cement Ratio: Nominal (1)	Water-Cement Ratio: Net (2)	Weight of Concrete, lb. per cu. ft.	Flow, per cent immediately after: Mixing	Flow, per cent immediately after: Remixing	Compressive Strength, lb. per sq. in.: 1 Day	7 Days	28 Days	28 Days[a]	1 Year
ELGIN SAND AND CHICAGO LIMESTONE—(Continued)										
0............	0.96	0.88	154	231	...	310	2040	3610	3080	6330
2............	0.96	0.88	153	231	192	360	2200	3740	3780	6420
3............	0.96	0.88	153	231	174	320	2110	4040	2930	6490
4............	0.96	0.88	153	232	153	290	1980	4030	3200	6440
5............	0.96	0.88	153	231	131	280	1970	3810	3140[c]	6240
6............	0.96	0.88	153	232	Crumbled	230	1900	3820	3020	5440
0............	1.04	0.96	153	262	...	240	1660	3550	2640	5730
2............	1.04	0.96	153	262	236	270	1770	3840	2630	6180
3............	1.04	0.96	153	259	230	240	1780	3700	2820	6080
4............	1.04	0.96	153	259	210	220	1820	4010	3030	6080
5............	1.04	0.96	153	260	175	190	1650	3640	3080	6000
6............	1.04	0.96	152	257	141	160	1450	3490	2500	6040
0............	1.12	1.04	153	278	...	200	1320	2690	2350	4700
2............	1.12	1.04	152	279	248	210	1440	2910	2370	5260
3............	1.12	1.04	153	278	246	190	1410	3310	2450	5770[c]
4............	1.12	1.04	152	276	233	180	1390	2980	2350	5440
5............	1.12	1.04	152	277	220	160	1540	3550	3060	5990
6............	1.12	1.04	151	276	179	130	1420	2920	2720	5280
ELGIN SAND AND CHICAGO BLAST-FURNACE SLAG										
0............	0.80	0.71	156	158	...	450	2500	4680	3720	7270
2............	0.80	0.71	155	159	131	490	2620	4650	4140	6810
3............	0.80	0.71	155	157	123	430	2420	4610	4010	6750
4............	0.80	0.71	154	157	Crumbled	420	2190	4100	3810	6420
5............	0.80	0.71	152	156	Crumbled	330	1900	3610	3060	5340
6............	0.80	0.71	145	159	Crumbled	260	1320	2570	2250	3340[c]
0............	0.88	0.79	154	216	...	300	2020	3610	3120	6140
2............	0.88	0.79	154	213	179	380	2290	4290	3810	7180
3............	0.88	0.79	154	214	168	390	2350	4590	4000	6620
4............	0.88	0.79	153	215	145	320	1980	3700	3340	6440
5............	0.88	0.79	153	215	129	290	1920	4150	3370	5790
6............	0.88	0.79	152	218	Crumbled	230	1520	3240	3040	5080
0............	0.96	0.86	154	249	...	250	1520	3130	2760	5320
2............	0.96	0.86	154	247	209	300	1660	3460	3220	5410
3............	0.96	0.86	154	246	199	300	1640	3870	3820	5750
4............	0.96	0.86	154	247	186	250	1800	4000	3240	6420
5............	0.96	0.86	153	251	171	230	1640	3740	3260	6180
6............	0.96	0.86	152	249	135	200	1250	3480	3260	5850
0............	1.04	0.95	153	273	...	210	1240	2660	2150	4850
2............	1.04	0.95	153	273	252	270	1300	2780	2260	4780
3............	1.04	0.95	153	271	244	210	1370	2840	2540	5150[c]
4............	1.04	0.95	153	271	233	200	1400	3040	2530	4950
5............	1.04	0.95	153	274	211	180	1290	3030	2490	5340
6............	1.04	0.95	152	273	171	160	1330	3120	2720	4920
0............	1.12	1.02	152	289	...	160	920	2040	1800	3980[c]
2............	1.12	1.02	153	290	272	160	900	2100	2020	3920
3............	1.12	1.02	153	287	264	130	1090	2270	1840	3970
4............	1.12	1.02	152	290	256	150	970	2440	2050	4410[c]
5............	1.12	1.02	151	291	242	150	1010	2420	2280	4240
6............	1.12	1.02	151	290	216	140	1060	2470	2160	3700

[a] Cylinders cured in air of laboratory.
[b] Average of 2 tests.
[c] Average of 3 tests

TABLE 6—EFFECT ON STRENGTH AND FLOW OF CONCRETE OF PREMIXING CEMENT-WATER PASTES FOR VARIOUS PERIODS BEFORE MIXING WITH AGGREGATE

(Data from Group 4)

For details of tests, see text and general notes accompanying tables.

Paste Mixed with Aggregate after hr. Indicated	Water-Cement Ratio		Weight of Concrete, lb. per cu. ft.	Flow, per cent immediately after final mixing	Compressive Strength, lb. per sq. in.				
	Nominal (1)	Net (2)			1 Day	7 Days	28 Days	28 Days[a]	1 Year
CONCRETE MIX BY VOLUME, 1:3									
0	0.72	0.64	154	243	500	3560	5960	5240	7950
2	0.72	0.64	154	220	440	2900	4640	4170	7290
3	0.72	0.64	154	211	440	2980	5070	4290	7180
4	0.72	0.64	154	204	420	2720	4990	4050	6820
5	0.72	0.64	154	169	370	2830	4580	3870	6710
6	0.72	0.64	154	136	320	2530	4150	3400	6060
CONCRETE MIX BY VOLUME, 1:4½									
0	0.96	0.85	155	258	260	1990	4120	3540	6490
2	0.96	0.85	154	240	220	1830	3620	2960	5160
3	0.96	0.85	154	232	230	1880	3680	2910	5490
4	0.96	0.85	154	224	220	1890	3650	3040	5620
5	0.96	0.85	154	211	200	1760	3410	2930	5250
6	0.96	0.85	154	205	150	1630	3280	2570	5340
CONCRETE MIX BY VOLUME, 1:7									
0	1.40	1.22	154	259	70	770	1760	1340	2760
2	1.40	1.22	154	248	70	750	1570	1290	2580
3	1.40	1.22	153	240	70	760	1620	1360	2650
4	1.40	1.22	153	234	80	800	1650	1310	2440
5	1.40	1.22	153	230	70	710	1550	1420	2500
6	1.40	1.22	153	215	50	610	1450	1170	2420

[a] Cylinders cured in air of laboratory.

TABLE 7—STRENGTH AND FLOW OF CONCRETE MADE WITH WET AGGREGATES AND REMIXED AFTER VARIOUS PERIODS

(Data from Group 5)

For details of tests, see text and general notes accompanying tables.

Remixed after hr. Indicated	Water-Cement Ratio		Weight of Concrete, lb. per cu. ft.	Flow, per cent immediately after		Compressive Strength, lb. per sq. in.				
	Nominal (1)	Net (2)		Mixing	Remixing	1 Day	7 Days	28 Days	28 Days[a]	1 Year
CONCRETE MIX BY VOLUME, 1:3										
0	0.72	0.64	155	247	...	650	3510	6210	4810	7945
2	0.72	0.64	155	245	219	620	3310	5910	4130	7990
3	0.72	0.64	154	242	210	560	3390	5900	4710	8020
4	0.72	0.64	154	244	186	500	3320	5500	5070	7950
5	0.72	0.64	153	242	146	480	3270	5480	4770	7970
6	0.72	0.64	153	243	116	400	2830	5060	4110	7500
CONCRETE MIX BY VOLUME, 1:4½										
0	0.96	0.85	155	244	...	290	1900	3750	2990	5900[c]
2	0.96	0.85	155	247	221	310	1890	3830	3520	6190
3	0.96	0.85	155	243	213	270	2110	4510	3580	6150[c]
4	0.96	0.85	155	245	202	250	2070	4370	3420	6660
5	0.96	0.85	154	249	188	210	2000	4240	3410	6170
6	0.96	0.85	154	248	157	180	1590	3310	2910	5700
CONCRETE MIX BY VOLUME, 1:7										
0	1.40	1.22	154	244	...	90	740	1590	1350	3050
2	1.40	1.22	154	248	222	90	720	1850	1250	2880
3	1.40	1.22	154	249	219	100	860	2100	1800	3230
4	1.40	1.22	154	246	217	80	720	1670	1210	2620
5	1.40	1.22	154	249	213	90	980	2210	1990	3870
6	1.40	1.22	153	244	179	60	780	1960	1680	3560

[a] Cylinders cured in air of laboratory.
[c] Average of 3 tests.

TABLE 8—STRENGTH AND FLOW OF CONCRETE CONTAINING CELITE OR HYDRATED LIME REMIXED WITHOUT ADDITION OF WATER AFTER VARIOUS PERIODS

(Data from Group 6)

Concrete mix: 1:4½ by volume.
Water-cement ratios based on:
(1) Nominal—amount of water used in first mixing.
(2) Net—nominal water corrected for absorption of aggregate *only*.
For details of tests, see text and general notes accompanying tables.

Remixed after hr. Indicated	Admixture		Water-Cement Ratio		Weight of Concrete, lb. per cu. ft.	Flow, per cent immediately after		Compressive Strength, lb. per sq. in.				
	Per Cent by Volume	Per Cent by Weight	Nominal (1)	Net (2)		Mixing	Remixing	1 Day	7 Days	28 Days	28 Days[a]	1 Year
						CELITE						
0.......	15	2.12	0.96	0.85	153	235	...	350[b]	2300[b]	4430[b]	3300[b]	6540[b]
2.......	15	2.12	0.96	0.85	153	238	206	360	2310	4440	3700	6580
3.......	15	2.12	0.96	0.85	153	237	192	320	2410	4470	3070	6420
4.......	15	2.12	0.96	0.85	152	237	175	280	2210	4480	3730	6380
5.......	15	2.12	0.96	0.85	152	238	137	250	2140	4200	3250	6280[c]
6.......	15	2.12	0.96	0.85	150	238	Crumbled	210	1740	3450	3100	4820
0.......	25	3.56	0.96	0.85	152	217	...	360[b]	2310[b]	4510[b]	3940[b]	6100[b]
2.......	25	3.56	0.96	0.85	152	217	186	370	2340	4450	3500	6470
3.......	25	3.56	0.96	0.85	153	215	174	340	2430	4590	3950	6470
4.......	25	3.56	0.96	0.85	152	215	151	330	2320	4490	3520	6290
5.......	25	3.56	0.96	0.85	151	214	Crumbled	290	2100	4280	3150	5630
6.......	25	3.56	0.96	0.85	145	214	Crumbled	180	1190	2410	2120	3960
						HYDRATED LIME						
0.......	15	6.00	0.96	0.85	153	242	...	360[b]	2450[b]	4260[b]	3630[b]	6150[b]
2.......	15	6.00	0.96	0.85	153	242	217	330	2350	4300	3510	6530
3.......	15	6.00	0.96	0.85	153	243	206	350	2500	4670	3750	6380
4.......	15	6.00	0.96	0.85	153	243	184	280	2300	4560	3830	6490
5.......	15	6.00	0.96	0.85	152	246	152	230	2150	4020	3480	6290
6.......	15	6.00	0.96	0.85	152	242	129	220	2030	3870	3130	5520
0.......	25	10.0	0.96	0.85	153	229	...	380[b]	2520[b]	4150[b]	3540[b]	5860[b]
2.......	25	10.0	0 96	0.85	153	229	198	360	2550	4350	3410	6560
3.......	25	10.0	0.96	0.85	153	228	189	370	2550	4530	4340	6420
4.......	25	10.0	0.96	0.85	153	227	163	300	2480	4450	3720	6590
5.......	25	10.0	0.96	0.85	152	226	136	280	2280	4140	3500	5990
6.......	25	10.0	0.96	0.85	146	223	Crumbled	210	1470	3110	2400	5410[d]

[a] Cured in air of laboratory.
[b] Average of 8 tests.
[c] Average of 3 tests.
[d] Average of 2 tests.

TABLE 9—STRENGTH OF CONCRETE CONTAINING CELITE OR HYDRATED LIME RESTORED TO ORIGINAL WORKABILITY BY REMIXING WITH ADDITION OF WATER AFTER VARIOUS PERIODS

(Data from Group 7)

Concrete mix: 1:4½ by volume.
Water-cement ratios based on:
(1) Nominal—amount of water used in first mixing.
(2) Final—nominal water plus that used to restore original workability.
(3) Net—final water corrected for absorption of aggregate *only*.
For details of tests, see text and general notes accompanying tables.

Remixed after hr. Indicated	Admixture		Water-Cement Ratio			Weight of Concrete, lb. per cu ft.	Flow, per cent immediately after		Compressive Strength, lb. per sq. in.				
	Per Cent by Volume	Per Cent by Weight	Nominal (1)	Final (2)	Net (3)		Mixing	Remixing	1 Day	7 Days	28 Days	28 Days[a]	1 Year
						CELITE							
0.......	15	2.12	0.96	0.96	0.85	153	242	...	350[b]	2300[b]	4430[b]	3300[b]	6540[b]
2.......	15	2.12	0.96	1.01	0.89	153	238	240	300	1960	4040	3170	6190
3.......	15	2.12	0.96	1.02	0.91	152	242	238	290	1810	3750	3030	5810
4.......	15	2.12	0.96	1.05	0.93	152	238	237	230	1780	3740	2720	5800
5.......	15	2.12	0.96	1.08	0.96	151	238	235	180	1480	3390	2630	5200
6.......	15	2.12	0.96	1.14	1.03	150	238	234	120	1160	2920	2550	4540[c]
0.......	25	3.56	0.96	0.96	0.85	153	216	...	360[b]	2310[b]	4510[b]	3940[b]	6100[b]
2.......	25	3.56	0.96	1.01	0.90	153	216	216	320	2020	4350	3260	6430
3.......	25	3.56	0.96	1.04	0.92	152	217	218	290	1840	3960	3260	5400
4.......	25	3.56	0.96	1.06	0.94	152	215	215	240	1710	3700	2920	5380
5.......	25	3.56	0.96	1.10	0.99	150	218	208	180	1470	3480	3050	5510
6.......	25	3.56	0.96	1.16	1.04	150	219	215	120	1220	2920	2660	4480
						HYDRATED LIME							
0.......	15	6.00	0.96	0.96	0.85	154	246	...	360[b]	2450[b]	4260[b]	3630[b]	6100[b]
2.......	15	6.00	0.96	1.01	0.89	153	244	245	300	1950	3770	3040	5440
3.......	15	6.00	0.96	1.02	0.90	153	245	244	260	2050	4070	3340	6230
4.......	15	6.00	0.96	1.04	0.93	153	247	243	200	1770	3590	3130	5410
5.......	15	6.00	0.96	1.10	0.98	152	246	241	150	1550	3290	2590	4950
6.......	15	6.00	0.96	1.16	1.04	151	246	241	100	1210	2730	2060	4620
0.......	25	10.0	0.96	0.96	0.85	153	229	...	380[b]	2520[b]	4150[b]	3540[b]	5860[b]
2.......	25	10.0	0.96	1.01	0.89	153	228	227	290	2110	3550	3010	5740
3.......	25	10.0	0.96	1.03	0.91	153	228	227	250	2040	3700	3180	5720
4.......	25	10.0	0.96	1.05	0.94	152	227	225	230	1820	3950	3010	5550
5.......	25	10.0	0.96	1.10	0.99	152	227	223	150	1520	3310	2210	4870
6.......	25	10.0	0.96	1.16	1.05	151	225	227	110	1220	2640	2070	4130

[a] Cylinders cured in air of laboratory.
[b] Average of 8 tests.
[c] Average of 3 tests.

TABLE 10—STRENGTH AND FLOW OF CONCRETE AS AFFECTED BY PREMIXING OF CEMENT-WATER-PASTE FOR VARIOUS PERIODS AND BY TIME OF MIXING

(Data from Group 8)

Concrete mix: 1:4½ by volume.
Nominal water-cement ratio 0.96 (0.85 when corrected for absorption of aggregate).
For details of tests, see text and general notes accompanying tables.

Time of Mixing, min.			Weight of Concrete, lb. per cu. ft.	Flow, per cent	Compressive Strength, lb. per sq. in.				
Cement and Water	Cement, Water and Aggregates	Total			1 Day	7 Days	28 Days	28 Days[a]	1 Year
0	¼	¼	154	263	300	2090	4160	3820	6140
0	½	½	154	262	320	1930	3920	3600	6070[b]
0	1	1	155	259	310	2070	4150	3320	6340
0	2	2	155	256	350	2160	4060	3520	6520
0	5	5	155	249	390	2440	4530	4060	6390
0	10	10	155	234	420	2540	4550	4300	6660
5	½	5½	154	249	260	1940	3840	3250	5940
10	½	10½	154	246	270	2000	3970	2960	6000
20	½	20½	154	242	270	1990	3740	3290	5850
30	½	30½	155	235	320	2170	4060	3600	5550
5	1	6	155	256	300	1930	4170	4010[b]	6060
10	1	11	155	253	340	2030	4000	3430	6010
20	1	21	155	245	310	2000	4010	3400	5750
30	1	31	155	237	290	2070	3990	3030	5250
5	2	7	154	254	390	2080	4090	3200	6130
10	2	12	154	252	310	2040	4200	3590	5980
20	2	22	154	244	360	2060	4180	3250	6190
30	2	32	154	242	370	2220	4300	3240	6080
5	5	10	155	247	330	2230	4260	3490	6120[b]
10	5	15	154	243	340	2250	4180	3770	5630
20	5	25	154	239	380	2150	4270	3150	6070
30	5	35	155	226	370	2260	4260	3460	6170
5	10	15	154	242	370	2430	4260	3790	6390
10	10	20	154	238	370	2500	4420	3580	6540
20	10	30	155	226	380	2510	4380	3810	6130
30	10	40	155	217	420	2580	4410	3680	6520

[a] Cylinders cured in air of laboratory.
[b] Average of 3 tests.

TABLE 11—TESTS OF CEMENT

All tests made in accordance with the Standard Specifications and Tests for Portland Cement of the A.S.T.M (Serial Designation C9-26)

CHEMICAL ANALYSIS

Lot No.	Silica (SiO_2)	Iron Oxide (Fe_2O_3)	Aluminum Oxide (Al_2O_3)	Calcium Oxide (CaO)	Magnesium Oxide (MgO)	Sulfuric Anhydride (SO_3)	Loss on Ignition	Insoluble Residue	Total
8645	21.80	3.46	6.07	63.00	1.94	1.94	1.21	0.13	99.42
8646	21.54	2.79	5.75	61.78	4.60	1.75	1.21	0.12	99.42
8647	21.89	2.23	6.53	63.60	2.20	1.60	1.28	0.08	99.33
8648	22.51	3.04	6.77	61.58	1.97	1.67	1.71	0.27	99.25
8649	21.93	2.88	6.28	62.49	2.68	1.74	1.35	0.15	99.35

PHYSICAL TESTS

Lot No.	Miscellaneous Tests					Tensile Strength 1:3 Briquets, lb. per sq. in.	
	Fineness, per cent Residue on 200-Mesh Sieve	Normal Consistency	Time of Setting: Initial h. m.	Time of Setting: Final h. m.	Soundness (over boiling water)	7 Days	28 Days
8645	13.2	23.0	3 15	7 00	OK	285	380
8646	11.5	23.5	3 20	7 35	OK	315	420
8647	15.0	23.5	3 25	8 00	OK	315	415
8648	14.6	23.5	3 05	7 05	OK	265	365
8649	13.6	23.5	3 10	7 35	OK	265	390

1818 Treatise on Mortars and Cements, by L. J. Vicat, 1818;

Translation from French by J. T. Smith, 1837, p. 79.
"There is not the least danger in beating up fresh mortar which has begun to harden, and bringing it back to its first consistency by the addition of water, provided this hardness was brought on in a few hours by heat of sun or scorching wind."

1885 Briquets Retempered after Pulverizing, by E. Clarke;

Transactions Am. Soc. C.E., v. 14, p. 169, 1885.

1886 Plastic Concrete, by W. R. Kinipple;

Proc. Inst. C. E., v. 87, p. 66, 1886.
Tests on retempered concrete.

1888 Experiments on Regauged Mortar, by H. Faija;

Soc. of Engr., London, 1888.
Test results given.

1893 Investigation on Effect of Retempering upon Strength of Cement Mortars, by Goddard and Evans;

Eng. News, v. 29, p. 16, Jan. 5, 1893.
Tests with 4 Portland and 2 Rosendale cements indicate that all cements lose strength by resetting. Rosendale was affected more than portland, former appeared to regain strength at faster ratio than latter. Retempering not recommended.

Regaging Cement Mortar, Cohesion and Adhesion, by E. S. Wheeler;

Report Chief of Engr., U. S. A., 1894, p. 2327; 1895, p. 2979; 1896, pp. 2814, 2840 and 2868.
Test results tabulated.

1898 Cements and Hydraulic Limes, by E. Candlot; p. 355.

Mortars regaged after 12 and 24 hr., while showing great loss in strength at 7 and 28 days, gave only small loss on long-time tests.

1901 Cements Held in Mixing Bed Different Periods before Setting Undisturbed;

Tests of Metals, Watertown Arsenal, 1901; 1904, p. 343. Minutes, Assn. Am Portland Cement Mfr., p. 7, April, 1905.
Compression tests on concrete from same batch at 2-hr. intervals up to 2½ days. Several domestic portlands, German portland and natural cement used. Portland cements maintain their strengths several hours after gaging. Strengths at various periods given.

1902 Recent Experiments with Cements, Mortar, and Concrete, by T. S. Clark;

Eng. News, v. 48, p. 67, July 24, 1902.
Tables of test results with tempered and untempered Rosendale and portland cements. The quicker-setting (Rosendale) more seriously affected by retempering. Retempering should not be permitted.

Retarding Setting of Cement by Continuous Mixing after Wetting, by G. Y. Skools;

Eng. News, v. 48, p. 382, Nov. 6, 1902.
Tensile strengths of cements and mortars with various times of mixing to determine how long concrete can be kept alive.

1903 Materials of Construction, by J. B. Johnson;

Published by Wiley, N. Y., 1903, p. 593.
Tests from various authorities described.

Mortar Experiments, by T. F. Richardson;

Report, Metropolitan Water and Sewerage Board, 1902, p. 93; 1903, p. 120.

1905 Retempering, by P. L. Wormeley, Jr.;

Farmers Bull. 235, U. S. Bureau of Public Roads, 1905, pp. 13 and 31.
Concrete suffers little loss of strength if thoroughly mixed with sufficient water to restore normal consistency. Table of test results.

Concrete and Concrete-Steel in United States—Retempering, by E. Thatcher;

Cement Age, v. 1, p. 330, Jan., 1905.
Mentions Clark and Skeels' tests and explains Kinipple system of placing concrete under water, which consists of allowing concrete to partially set before placing, so that cement will not separate when thrown into water.

1906 Retempered Mortar in Concrete Work, by E. McCullough;

Eng. News, v. 55, p. 30, Jan. 11, 1906.
Abstract, *Cement Age*, v. 2, p. 678.

Tempering Mortar, by J. H. Granberg;

Eng. and Min. Jour., v. 81, p. 88, Jan. 13, 1906.
Mentions tests of C. W. Smith on tempered concrete. Suggests that concrete made of portland cement be placed before setting.

1907 Cement and Concrete, by L. C. Sabin;

Published by McGraw-Hill, N. Y., 1907, pp. 251 and 259.
Tabulates test results. Adhesive strength of mortars diminished by regaging. Regaged mortars give good results in sea water.

Concrete and Reinforced Concrete Construction, by H. A. Reid;

Published by Clark, N. Y., 1907, p. 133.
Tests with German and domestic portland and natural cements.

Process of Preserving Completely Prepared Cement Mortar without Loss of Its Property of Setting, by J. H. Magens;

French Pat. 373,834, Jan. 23, 1907.
Jour. Soc. Chemical Ind., v. 26, p. 762, July 15, 1907.
Mortar cooled at time of manufacture below temperature of air and kept in movement or shaking until used, in order to retard setting.

1909 Influence of Delay in Application of Cement Mortar, by H. Urbach;

Zement u. Beton, p. 392, June, 1909.
Tables show test results with retempered mortars at various periods after gaging.

Influence of Storing Gauged Cement Mortar, by H. Burchartz;

Mitt. Kgl. Materialprüf., v. 4, 1908.
Cement Age, v. 9, p. 36, July, 1909.
Strength test results at Gross Lichterfelde; storing up to 8 hr. does not materially affect strength of slow-setting mortars.

Bonding of Layers of Mortar after Different Time Intervals, by B. Kirsch;

Proc. Int. Assn. Testing Mat., No. XIII, 1909.

1911 Concrete Methods in Rochester;

Municipal Jour., June 7, 1911.
Experiments showing strength increase and deformation decrease by retempering.

Retempered Mortar as Bonding Material, by J. F. Skinner;

Municipal Jour., May 31, 1911.
Cement Age, v. 13, p. 97, Sept., 1911.
Recommends slush coat of retempered concrete on old concrete before new concrete is poured as expansion due to sudden rise in temperature and decrease in volume due to loss of heat in initial setting is eliminated and thus there is no tendency for new and old concrete to shear along joint.

Effect of Regaging Mortar, by H. Burchartz;

Mitt. Kgl. Materialprüf., v. 29, p. 164, 1911.
Jour. Soc. Chemical Ind., v. 30, p. 687, 1911.
Diagram and tables to show decrease in strength in proportion to time which elapses between gaging and regaging.

1913 Retempered Mortar as Finishing Coat for Concrete Surfaces, by J. B. Hawley;

Eng. News-Rec., v. 69, p. 689, April 3, 1913.
Method of retempering and applying mortar.

Retempered Mortar as Bond between Old and New Concrete, by A. A. Northrop;

Eng. News, v. 70, p. 178, July 24, 1913, p. 274, Aug. 7, 1913.
Practice in British Isles to use small amount of retempered mortar in setting of tile in wall. This prevents cracking, as mortar holds its shape. Method of mixing described.

1916 Concrete, Plain and Reinforced, by Taylor and Thompson;

Published by Wiley, N. Y., 1916, p. 173.
Tests indicate that cement mortar or concrete can be regaged without injury up to time of initial set. Thereafter strength is reduced. Regaging of mortar that has begun to stiffen should not be permitted.

Effect of Prolonged Keeping of Concrete in Wet Condition on its Subsequent Shrinkage on Hardening in Air, by M. Rudeloff;

Mitt. Kgl. Materialprüf., v. 34, p. 2, 1916.
Jour. Soc. Chemical Ind., v. 35, p. 1110, 1916.

Central Concrete-Mixing Plant Tried at Baltimore, by G. Y. Carpenter;

Eng. News, v. 76, p. 268, Aug. 10, 1916.
Operation of plant where concrete was delivered for sale. Deliveries made within 30 minutes after leaving mixer or batches rejected. Projects constructed by this method successful. Delivered in contractors' hydraulic-dump-body type of motor truck. Tests at Meade Laboratory, Baltimore, show that jigging mortar for periods up to 1½ hr. does not materially decrease strength of concrete; small amount of shaking seemed beneficial.

1918 Concrete Engineers Handbook, by Hool and Johnson;

Published by McGraw-Hill, N. Y., 1918, p. 232.
Results of Candlot, Sabin, and McCullough summarized.

Treatise on Masonry Construction, by I. O. Baker;

Published by Wiley, N. Y., 10th Edition, 1918, p. 100.
Retempering makes mortar easier to handle. Increases time of setting and usually weakens cement mortar. Retempered mortar not recommended for use.

1920 Premixing No Benefit in Concrete;

Eng. News-Rec., v. 84, pp. 335, 473, Feb. 12 and Mar. 4, 1920.
Bureau of Standards tests show no advantage on strength of concrete in premixing cement and water before (15 to 30 min.) adding aggregate.

1921 Strength of Concrete at Increasing Intervals after Mixing, by R. C. Durst;

Eng. News-Rec., v. 87, p. 773, Nov. 10, 1921.
Cement and *Eng. News*, v. 34, p. 27, 1922.
Compressive and tensile tests on remixed, tempered, and concrete allowed to stand for a number of hours after mixing, at Univ. of Akron, show very little strength is lost up to the 3rd or 4th hour and an increase in strength is not noticeable after the 4th hour of mixing.

Strength Tests of Pre-Mixed Concrete on Detroit Street Railways, by W. R. Dunham, Jr.;

Eng. News-Rec., v. 87, p. 681, 1921.

Retempering Cement Mortar;

Brick—How to Build and Estimate, by W. Carver, 1921, p. 39.
Retempering seriously decreases strength of quick-setting mortars.

Central-Plant-Mixed Concrete Tested for Maximum Safe Haul;

Public Roads, Dec., 1921, p. 22.
Slump and compressive strength of concrete hauled in truck up to 3½ hr. Slump decreased with time; strength not changed.

1922 New Developments in Concrete Mixing;

Eng. and Ind. Manage., v. 17, p. 441, May 18, 1922.

1926 Studies on Duration of Set and Remixing of Concrete, by R. Feret;
Rev. Mat. des Const. et Trav. Pub., No. 203, p. 237, 1926.
Concrete mixtures of 7 and 9% water remixed 1½, 5 and 8 hr. after mixing.

Prehydration of Cement in New Method of Concrete Mixing, by W. B. Jones;
Eng. News-Rec., v. 96, p. 850, May 27, 1926.

1927 New Method of Mixing Concrete;
Ferro-Concrete (London), Jan., 1927.
U. S. patent provides for mixing concrete in 2 stages. In first stage cement is mixed with the water and in the second, the grout and ingredients are mixed together. (15 seconds for each mixing.) Prevents balls of unhydrated cement, saves time in mixing and makes economical plant layout.

1928 Strength Characteristics of Concrete as Indicated by Core Tests, by A. N. Johnson;
Public Roads, v. 9, p. 148, Sept., 1928.

1929 Commercial Central Mixing Plants for the Manufacture and Sale of Ready-Mixed Concrete, by Norman Beggs;
Concrete, v. 34, p. 37, Feb., 1929.
Growth in number of central mixing plants; selection of site; construction of plant; provisions for cold weather; bulk cement; tests on hauled concrete.

Discussion—Tests of Retempered Concrete

Mr. Gilkey. HERBERT J. GILKEY*—At a time when the trend in concrete practice seems to be so definitely toward the more extensive use of the central mixing plant with the added importance of the elapsed time between mixing and placing, the content of this paper is of more than usual pertinence.

In addition it should contribute greatly toward removing a confusion of long standing as to just what is to be considered as objectionably retempered concrete.[1 2 3]

It is quite evident that usual specifications instead of being merely "safe" have been rather unreasonable regarding the permissible time from mixing to placing.

Time Elapsed Between Mixing and Placing—Slater and Walker[4] reported a distinct gain in strength (as much as 25 or 30 per cent) in samples that were not molded until an hour after original mixing. This held true even when evaporation was prevented by storage in a tight moist box. In the spring of 1925 the writer undertook some similar experiments in the concrete laboratory at the University of Colorado.

Eight batches of 1:2:3 concrete (absolute volume) with a water-cement ratio of 0.65 were prepared. Time was taken from the moment that water was added to the batch. All batches were mixed thoroughly by hand and each in turn was set to one side. The last to be mixed was made into 6 x 12-in. cylinders at once. The others followed in the reverse order to that in which the water was added. Thus the first batch to be mixed was the last one to be re-mixed and placed. All batches were stiff (from 0 to 1 in. slump) but were workable and properly placed by thorough tamping. Batches were weighed when first set side and were re-weighed at the re-mixing. This gave the evaporation loss, which proved to be very slight. The series was repeated on three different days for batches in which no correction was made for the loss and three other series were run in which the loss was made up by the addition of the right amount of water at the re-mixing. The time between mixing and placing ranged from 9 min. to 207 min. (about 3½ hr.), this being the time elapsed from the addition of water to the batch to the completion of the specimen. Slump tests were not repeated at the re-mixing but the appearance indicated that there was little change in workability. Much change would have made this dry mixture very unworkable. The results

* Associate Professor of Civil Engineering, University of Colorado.
[1] A. R. Lord, *Proc.* A.C.I., 1927, p. 46.
[2] D. A. Abrams, *Proc.* A.C.I., 1926, p. 622.
[3] P. H. Bates, *Proc.* A.C.I., 1926, p. 623.
[4] *Proc.* A.S.C.E., Jan., 1925, Figs. 23, p. 33, and Table 20, p. 57.

indicated no evident effect upon strength. The maximum spread was from 4500 to 5500 lb. per sq. in. at 28 days standard curing for individual specimens. This maximum spread occurred at about the same time interval and the mean strength, which was an average from six specimens, was about 4900 lb. per sq. in. for all periods. The evaporation correction was so slight that these batches gave results practically identical with those from the batches without evaporation correction. That is, the change in water-cement ratio was negligible for the conditions of these tests.

These findings did not accord with those of Slater and Walker, but are in close agreement with those of Gonnerman and Woodworth, whose tests cover a longer range of elapsed time than did these. There is nothing in any of the three sets to indicate any harmful results from a considerable time lapse.

Between 1921 and 1928 incidental excursions have been made into similar territory from time to time, and in various connections mixtures have been continuously agitated until placed, remixed at intervals, etc. On at least one occasion it was possible to delay initial stiffening to about 8 hr. after original mixing. The exact period that this can be done is likely to be a function of the cement itself, the dryness and temperature of the air and the wetness of the mix. (The wet mix stiffens more slowly.) From all these reconnaissances, it appears very reasonable to conclude that the critical period will be clearly indicated by the stiffening of the mixture and that that is the feature to be closely watched in this connection. Had the Colorado tests extended over 5 or 6 hr., instead of a 3½ hour maximum, evaporation would have been a more important item. In any case it would be so in dry weather out of doors where there is a greater air circulation.

Retempering When the Cement Takes Flash Set from Air Exposure—This is only remotely connected with the theme of the paper but may contribute something of value. Several years ago pans of cement were exposed to the air of the laboratory for about a week. When this cement was mixed, the mortar stiffened very quickly (within 10 min. or so after the water was added). There was a distinct evolution of heat, similar to the early setting stage for plaster of paris. To make the mass at all workable considerable water had to be added and even then it was lumpy and unsatisfactory. The specimens were much weaker than those made from properly stored cement of the same lot. What the air exposure did to the cement the writer does not know. The short-time mixing symptoms were not greatly different from those that appear after several hours have elapsed in such tests as the ones under discussion, except for the noticeable heat generated. The writer has followed this phase no further and it is possible (though not probable) that the phenomenon was limited to the local cement used. If general, it illustrates the necessity for keeping the cement protected from ordinary air exposure as well as the probable impossibility of getting satisfactory concrete from mixtures that have started to stiffen.

The Pre-wetting of Aggregates—In 1923 the writer participated in tests at the University of Illinois in which about 30 different types of coarse aggregate were used. These were added to the batch in three conditions: (*a*) dry; (*b*) after 15 min. soaking; (*c*) saturated. Some were very porous and when added dry they extracted much moisture from the batches. Among the usual types of aggregate, the dry or wet condition had little effect if proper allowance was made for the water carried in on the surface of the aggregate. The usual gravels and crushed stones have a very low total absorption but what absorption does take place, occurs mainly in the first few minutes of immersion. Fifteen minutes of soaking is, for these stones, the approximate equivalent of saturation. Sandstones and some limestones are exceptions to this rule. This agrees with conclusion number 4 of the paper, although the added workability was not noted.

Time of Mixing—This is a phase on which the information available is not in full agreement. A striking thing about these tests is the great increase in strength gained by mixing for 10 min. over that for 5 min. This shows especially in the 7-day strengths of Fig. 8. The periods above 2 min. mixing have generally indicated only a moderate gain in strength at a decreasing rate as the period was prolonged. The effect on early strengths is supposed to be relatively much greater[1] than on later ones, which is borne out by Fig. 8 if the 1-day strengths be excluded. It seems reasonable to exclude these since the setting process is hardly under way at the 1-day age. Three-day tests might have been illuminating.

The Air-Cured 28-Day Specimens—An interesting feature of these tests was the inclusion in every series of duplicate 28-day specimens that were cured in the air of the laboratory and tested dry. These appear in all the tables.

The strengths of the air-cured specimens are below those that were standard cured (moist cured and wet at test) as is to be expected. The comparative tabular values do not, however, tell more than about half the story. The misconceptions regarding moist curing in its relation to strength are still sufficiently widespread to warrant some additional comment on these data. In comparing the two columns of 28-day strengths some persons are likely to conclude that the absence of moist curing isn't such a serious matter after all. The data as given need to be compared with the following facts clearly in mind.

(*a*) The air-cured specimens were dry at test while the standard cured specimens were saturated. If the standard cured specimens had been air dry at test (exposed to dry air for from 10 to 20 days, depending upon the humidity or dryness of the air) their strengths would have been increased from 15 to 30 per cent. Or if the dry air-cured specimens had been saturated by immersing in water for from 12 to 24 hr. before test their strengths would have been decreased from 15 to 30 per cent. In either case the difference in strength would have been about twice that which was given by the tests as made. When the strengths of two con-

[1] *Proc.* A.C.I., 1928, Table 1, p. 166.

cretes are being compared it is essential that they be in the same condition at test, as regards the wet-dry condition. In this connection it may be added that saturation is the more satisfactory since it can be attained in a few hours and the added curing that results from the wetting is negligible. To dry in air is slow and rather indeterminate because of differences in humidity. Moist curing continues until the shortage of free moisture for continued hydration slows down or halts the curing process. It is therefore apparent that the concrete will take on added strength from curing as well as from mere drying during a 10 to 20-day air exposure after moist curing for 28 days. If removed from water earlier, the amount by which normal curing has been hampered is still indeterminate. Accelerated drying such as placing in an oven does not affect the concrete in the same way as air drying. The concrete is weakened by the forced drying and the strength gain of the dry condition over the wet will be partly, and sometimes entirely, neutralized by the weakening effect of the oven.

(*b*) As has been hinted above, air-cured concrete will vary greatly in strength at different places or at the same place at different times since the relative dryness or humidity of the air varies over a wide rage. When concrete is mixed, the water required to lubricate the mix greatly exceeds that necessary for hydration purposes (i. e. curing). If the concrete is exposed to the air this excess water evaporates. At first, curing proceeds at full speed, then as water for hydration becomes scarce the hydration slows down. Soon it is compelled to cease and the concrete remains in a more or less neutral state, i. e. at about constant strength. If the air be quite dry and the exposure long there is likely to be a falling off of strength, but that element may be ignored in the present discussion.

From the above, it is evident that a small body of concrete exposed to dry air will have its curing halted at a much earlier age than a large mass or concrete that is exposed to air that is more humid. In Chicago the air is usually rather humid. In a basement laboratory in Chicago it is probably quite humid. It is also still. Judging from some of the published data from P. C. A.-Lewis Institute tests made several yers ago,[1] it appears that the curing of 6 x 12-in. concrete cylinders is not halted in less than about 3 weeks. In this event the reported air-dry specimens might be expected to approximate 21-day specimens in strength if in the same condition at test. The concrete laboratory at the University of Colorado, while located in a moist basement is nevertheless much drier since the relative humidity at this altitude is much lower than in the vicinity of Chicago. An air-cured 28-day 6 x 12-in. specimen usually approximates the strength of a moist-cured 1 or 2-week specimen. If the concrete were subjected to circulating air such as would be encountered in outdoor exposure the rate of drying would be much greater both in Illinois and Colorado.

(*c*) Comparison for periods as short as 28 days are misleading. Under favorable curing, concrete continues to gain strength for an indefi-

[1] *Proc.* A.C.I., 1926, Fig. 11, pp. 421 and 422.

nite period. On the other hand the air-cured concrete does not gain strength appreciably as long as the dry air exposure continues after drying has advanced far enough to halt curing. Thus at greater ages the differential in favor of moist curing continues to increase.[1,2] Comparisons of curing conditions based upon periods of 28 days or less have been a source of much misleading inference.[3,4]

In the present case there is no question as to Mr. Gonnerman's and Mr. Woodworth's correct understandings regarding the points that have been raised. The injection of air-cured 28-day strengths was aside from the main purpose of the paper and probably did not, in their estimation, warrant the discussion necessary to fully safeguard them from being subject to misinterpretation. The writer, on the other hand, considers the points mentioned to be of the utmost importance in their bearing upon many forms and phases of concrete construction. Moreover the fallacies based upon them are so universal and harmful that if such data are to be shown at all, he feels that they should be very thoroughly explained. The writer has seen no more clear-sighted presentation of data on the strength effect of different conditions at test than is to be found in another paper in this volume of the proceedings. It is that by Mr. Gonnerman's and Woodworth's co-worker, Raymond Wilson. Mr. Wilson's Table 11 is especially illuminating. Some of the points briefly mentioned in this discussion are treated more fully in connection with a discussion of Mr. Wilson's paper.

To summarize:

(1) The data presented are unusually timely because of their bearing upon: (*a*) the problems of the central mixing plant, (*b*) the long-standing confusion as to what is meant by retempered concrete and, (*c*) the injustice of many current specifications as regards time allowed from mixing to placing.

(2) Less extensive experiments made by the writer in the spring of 1925 for a period up to over 3 hr. bear out the conclusion that the strength of the concrete is unaffected by elapsed time of at lest that duration if the concrete has not started to stiffen and the water-cement ratio has not been appreciably altered by evaporation, leakage, or absorption by the aggregate.

(3) Stiffening in a manner that is objectionable may be due to too much elapsed time or it may be from flash set due to a fault of the cement. In some reconnaissance tests by the writer, cement exposed in trays to the air of the laboratory for a week gave flash set and could not be properly worked for long even with the addition of considerable extra water. The resulting concrete was weak at all test ages.

(4) Tests by the writer in 1923 verify the conclusion that the prewetting of the aggregate has little effect if surface moisture and absorption be allowed for.

[1] *Proc.* A.C.I., 1926, Fig. 13, p. 424.
[2] *Trans.* A.S.C.E., Vol. 91 (1927), p. 161, Figs. 7 and 8.
[3] *Trans.* A.S.C.E., pp. 153–67.
[4] *Proc.* A.S.T.M., 1927, Part II, pp. 424–27.

(5) The time-of-mixing results are a contribution to this phase. Added experimental work on this subject seems to be needed to supply more perfect accord between existing reported data.

(6) In studying and comparing the results of the air-cured 28-day specimens that are listed in all the tables several points need to be kept in mind:

(*a*) The wet or dry condition at test is an important variable in its effect on strength. Strength comparisons in which any other variable occurs should be on specimens in the same condition at test. Saturation seems best because of the ease and certainty with which it may be attained. Water is about as wet one place as another. Air varies greatly. Oven drying has a disturbing effect upon strength.

(*b*) The rate of drying is dependent upon: (1) surface-volume relation of concrete (size and shape of specimen); (2) humidity of air (which varies greatly at different places and at one place at different times); (3) circulation of the air. The equivalent period of favorable curing is therefore a function of the three above mentioned variables. The results reported are much more favorable to the air-cured specimens than they would probably have been for smaller specimens cured in drier air or in air that had greater circulation.

(*c*) In comparing air-cured concrete with that which has been continuously moist cured (standard cured) the greater the age, the more favorable is the comparison likely to be to the standard curing since moist concrete continues to gain strength. Dry concrete is stationary or practically so. Its strength may even fall off under some conditions.

The paper as a whole has met a genuine need in supplying convincing data that should have a marked and immediate effect upon important phases of concrete practice. The writers are to be congratulated upon a worthy work well done.

H. F. GONNERMAN—In his discussion, Prof. Gilkey has touched upon a number of important points which were not covered in our paper. His general remarks on curing and on the strengths of the air-cured specimens at 28 days which were reported in the various tables are particularly pertinent. As he has stated, the injection of these 28-day strengths of air-cured specimens was aside from the main purpose of the paper and in making comparisons between the strengths of these air-cured, dry specimens and those of the moist-cured specimens tested wet the precautions pointed out by Prof. Gilkey should be given due consideration. **Mr. Gonnerman.**

Tests covering many of the points regarding curing and testing raised by Prof. Gilkey are now under way in our laboratory. They confirm in the main the statements he has made, and which he very correctly considers to be of the utmost importance in their bearing on many forms and phases of concrete construction.

A. S. DOUGLASS—I wish to ask whether, in premixing of the cement and water, there was any particular shape to the curve? **Mr. Douglass.**

H. F. GONNERMAN—You will observe from the left-hand group of diagrams in Fig. 7 that as the time of standing increased there was a **Mr. Gonnerman.**

consistent reduction in the compressive strength of the concrete which amounted to about 20 per cent for the 6-hr. standing period, where the cement and water was premixed 1 min. and then held for 6 hr. before adding to the aggregates in the mixer. The flow of the concrete from premixed paste was generally appreciably greater than that of the concrete made by the usual method of mixing and held for like periods.

Prof. Bauer. E. E. Bauer—Did you have considerable segregation of the paste when it was standing?

Mr. Gonnerman. H. F. Gonnerman—These pastes were all fairly wet, because the water-cement ratios of the concrete ranged from 5½ to 10 gal. of water per sack of cement. We thoroughly mixed the cement and the water, but there was some segregation in the pastes upon standing.

Prof. Bauer. E. E. Bauer—Did you continually stir them?

Mr. Gonnerman. H. F. Gonnerman—No, the pastes were put in the cans where they remained until dumped in the mixer, but the aggregate was in the mixer at the time the pastes were added.

Mr. Foster. A. Foster—Tests have recently been made in Philadelphia under field conditions with the mixer being operated up to a period of approximately an hour, samples being taken at intervals of 1½ min., 15 min., 30 min., etc. The results are identical with the results under laboratory conditions. However, at approximately an hour's period, the concrete became so glutinous that it could not be taken from the mixer.

At the beginning the slump was approximately 9 in. At one period certain results were obtained with a 3-in. slump. When mix became so stiff and pasty that it would not come out of the mixer, enough water was added to secure a 3-in. slump and the results of the tests at that time were practically identical with the original 3-in. slump tests.

The next day 3 cu. yd. of concrete were hauled through streets for a 7-hr. period. After about 4 hr. no slump was obtained. The workability of the concrete had been practically destroyed by the evaporation of the water from the top of the truck. Upon restoring the workability to any given slump, the resulting strength was practically that obtained by the previous strength at an earlier period with the same slump. These results are mentioned because they were obtained in practical tests under field conditions in cold weather.

Mr. Abrams. Chairman Abrams—It is a very interesting commentary showing that, after all, the discrepancy between laboratory and field is not so great as we are sometimes told. The era of truck-mixed concrete and longer distance of transportation is undoubtedly with us. Anything that throws light on this subject is certainly welcome in the *Proceedings* of our Institute.

Mr. Conahey. George Conahey—I just wanted to point out that workability, as discussed in this paper, is different from the workability which was discussed at this convention a year ago. In other words, the workability discussed here is measured by the flow table, which measures a change due to a change in water content. The increase in flow naturally indicates a change in workability. We need more information about what

workability really is before we get into a further discussion of what we all mean by workability.

H. F. GONNERMAN—There is considerable discussion about the term workability at the present time, and in this paper we did not mean to compare different mixes of the same flow and say that because they gave the same flow, they were necessarily of the same degree of workability. We used the flow table merely to measure the stiffening of the mass as the standing period increased, and in several cases we have called that the change of workability. This may not be strictly accurate, but I think that for a given mix and a definite method of treatment the change in flow does give some indication of the change in workability, although it may not be an absolute measure. Mr. Gonnerman.

A MEMBER—In regard to this practice of tempering specimens by making a mortar and allowing it to stand and then retempering by mixing on the top to get away from shrinkage, I wonder if the author of the paper observed any difference in shrinkage of the concrete placed after retempering? A Member.

H. F. GONNERMAN—We did not make shrinkage measurements on these concretes. Our experience indicates that retempering has no effect on the shrinkage after the concrete has assumed a definite structural condition. During the plastic period, however, there is a decided reduction in shrinkage, as evidenced by the beneficial effect of retempering in capping material for cylinder tests. Mr. Gonnerman.

DUFF A. ABRAMS—I think one thing we might readily conclude from these investigations and others of a similar nature is that the old idea that concrete which has been held 30 minutes is no good, certainly must be discarded. In fact, the evidence seems to indicate that holding concrete for one or two hours under proper conditions may actually improve it. Mr. Abrams.

Concrete Studies at the Bull Run Dam, City of Portland, Oregon

By T. C. Powers*

Synopsis

The problem of estimating the strength of concrete containing aggregate too large for the test specimen is studied.

In addition to the water-cement ratio, two more variables are introduced, (1) the paste content (paste is cement plus water) and, (2) the physical condition of the paste at the time of hardening, *i. e.*, its homogeneity.

The latter variable is shown to influence permeability as well as strength.

The dependence of the water-cement-ratio-strength relation upon the other two variables is discussed, and conditions are cited under which it does not exist at all.

The modulus of elasticity appears to be greatly influenced by the paste content.

Introduction

The Bull Run dam is the major unit of the storage project of the Bureau of Water Works, City of Portland, Oregon. The structure is of curved gravity type, about 200 ft. high and 1000 ft. long and contains over 200,000 yd. of plain concrete. Aggregate was secured from the river bars below the dam site and was screened and recombined so as to produce a uniform grading up to 9½-in. (later reduced to 7-in.), all of which went through the mixers. On this project the writer wa,s in charge of concrete production and control under the direction of B. E. Torpen, superintendent of design and construction.

Encouraged by the cooperation of Ben S. Morrow, chief engineer and Mr. Torpen, the writer devoted considerable time to the study of concrete in its general aspects as well as in its relation to our local problem. It is with the results of these studies that this paper deals. All experimental work was done in the field laboratory on the job.

One of the first problems confronted was the estimation of the strength of our concrete from the test results. Our routine practice was to bring a large sample of the mix to the laboratory where all but that passing a 2-in. sieve was rejected. This eliminated over 40 per cent of the batch. Our first efforts were to find the relation between our test results and the actual strength of the concrete but the work gradually

* Bureau of Water Works, Department of Public Utilities, Portland, Oregon.

expanded to a study of the effect of aggregate in general. Since our laboratory was equipped with a testing machine having a capacity of only 100,000 lb., direct tests on larger specimens was impossible. We, therefore, resorted to an indirect method.

Wet Screening Method

By removing successively greater portions of the aggregate by use of a series of screens and casting the cylinders from each remainder, we could get a group of specimens from the results of which we hoped to be able to predict the effect of aggregate in sizes and quantities greater than we could test directly. For example, starting with a quantity of concrete containing aggregate graded from zero to 3-in. we first carefully sampled the batch and cast a cylinder containing the whole mix. Then, we would pass the remainder through a 1½-in. screen and cast a cylinder from that which passed rejecting all aggregate retained on the screen. The ½-in. remainder would then be screened with a ¾-in. screen in the same manner, and so on, until such a batch would yield a series of cylinders made from concrete passing screens as follows: 3-in.; 1½-in.; ¾-in.; ⅜-in.; No. 4; No. 8. In most cases only the first four divisions were made.

Note that this method is directly analagous to our routine sampling procedure. Such treatment yields portions having practically identical water-cement ratios, and since the total mix was always plastic, and having a 4- to 5-in. slump, screening would not appreciably alter the voids-cement ratio. However, the maximum size and fineness modulus of the aggregate would be altered and the percentage composition of the cylinders would be different, the aggregate content becoming less as the maximum size was decreased. In working with the data it was found to be more convenient to deal in terms of the paste content instead of the aggregate content. The aggregate content is practically $1 - p$, where p is the paste content. The term "paste" refers to cement plus water, absolute volumes per unit volume of concrete.

In our study of concrete we could find no reason inherent in the concrete itself for regarding a part of the mix as "mortar." We, therefore, treated the total aggregate as a unit in the concrete but divided it into several divisions prior to mixing for convenience and accuracy in proportioning. All aggregate was graded according to the equation

$$p = 1 - \frac{(d)^n}{D}$$

where p = amount retained on sieve of opening d
d = size of clear opening
D = maximum size
n = a constant exponent whose value depends upon the fineness modulus desired.

The results of 28-day tests on cylinders made by this method are given in detail in Table I and are typified by the following tabulation:

Size	0–⅜-in.	0–¾-in.	0–1½-in.	0–3-in.	0–6-in.
Real mix	1:2.5	1:3.2	1:4.8	1:6.3	1:8
Fineness modulus	3.60	5.20	5.85	6.80	7.40
Water ratio	1.20	1.20	1.20	1.20	1.20
Paste content	0.50	0.40	0.30	0.24	0.18
Slump	10-in.	8-in.	6½-in.	4-in.	2½-in.*
Strength	2100	1930	1680	1500	1300*

* By calculation.

In analyzing the data a very definite relation was found between the paste content p and the strength S at any constant water cement ratio, w/c. The relation was found to be of the form

$$S = \frac{Ap^n}{B^{w/c}} \qquad (1)$$

Where:

S = compressive strength, lb. per sq. in.
A, B and n = constants.
p = paste content, which is defined as the sum of the absolute volumes of cement and water per unit volume of concrete.
w/c = water-cement ratio, cu. ft. of water per sack of cement.

For our data the equation becomes

$$S_{28} = \frac{17000p^{0.5}}{4.2^{w/c}} \qquad (2)$$

As shown by Table I, Col. 6, Eq. 2 gave an average residual of 165 lb. per sq. in. This covers values of p from 0.20 to 1.00, and values of w/c from 0.45 to 1.40.

This is not considered to be a very close agreement but considering that these tests were made in temporary quarters where curing conditions were not well controlled, and that most of the values represent but one test, the agreement is remarkably good.

Applying Eq. 2 to our local problem indicates that our total mix is from 10 to 15 per cent weaker than our test results.

PREMIXED-PASTE METHOD

While Eq. 2 seemed to furnish as good an answer to our first question (the relation of test strength to actual strength) as we could hope to get in our field laboratory, the general question of the effect of aggregate, or paste content, remained unsettled, for our equation could have been couched in terms of maximum size, or fineness modulus, instead of paste content with just as good results when applied to our wet screening method tests.

In order to determine which of the three variables, maximum size, fineness modulus, or paste content, caused the variation in strength at constant water-cement ratio, we resorted to the premixed-paste method.

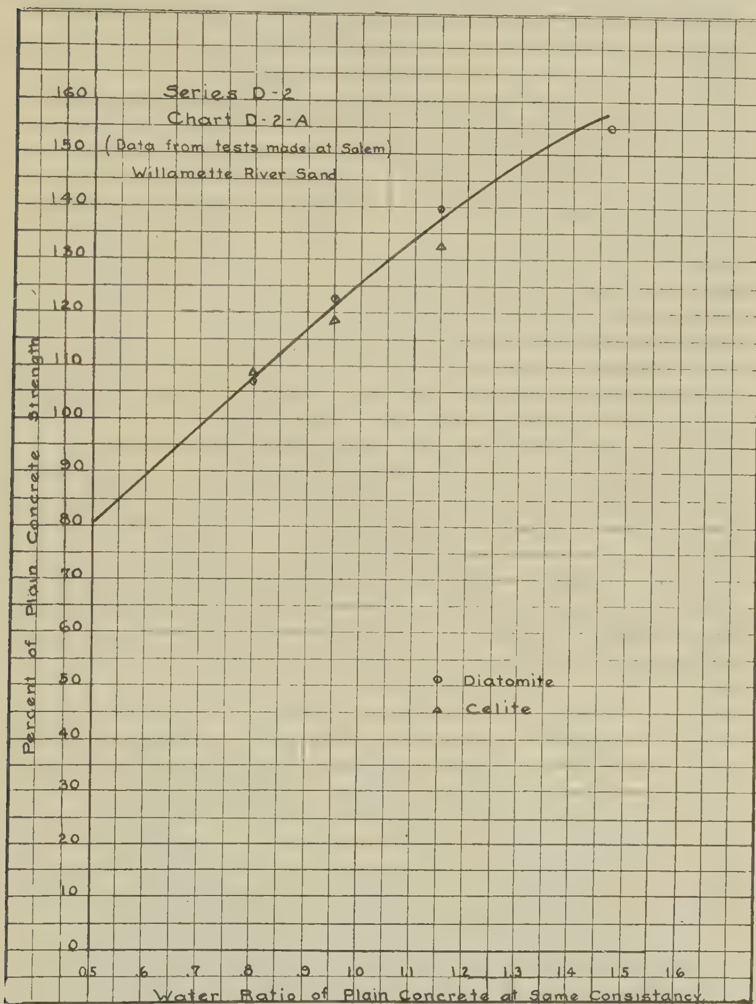


FIG. A.—DIATOMACEOUS EARTH TEST MADE IN OREGON STATE HIGHWAY DEPARTMENT LABORATORY.

This method consisted of adding various amounts and gradings of aggregate to a quantity of cement and water previously mixed in the desired ratio. In order to have a paste which would remain practically homo-

geneous while being handled, that is, one in which the cement would not settle out leaving free water on top, a water ratio of 0.635 was used.

This method differes from the wet-screening method in that it allows the paste to be handled separately, making it possible to vary the paste content without changing the characteristics of the aggregate.

Table II gives results of tests by the premixed-paste method. Three series were run covering the following conditions:

(1) Maximum size and fineness modulus, variable; consistency and water-cement ratio, constant.

(2) Maximum size and fineness modulus constant; water ratio constant; consistency varies with paste content.

(3) Using single-size (ungraded) aggregate, water ratio constant.

Due to the limited capacity of our testing machine, cylinders of this series were broken at the age of 2 days. Incidentally, this proved to be sooner than necessary but other tests have shown that time does not lessen the effect. It will be noted that in plastic mixtures, the effect is practically the same in all three cases. This is indicated by the constancy of the ratio $\left(\frac{S}{p^{0.5}}\right)$ (Col. 5), the variation from the average value of 1908 being small. Here, too, each value represents but one test.

Supplementing these tests were tests of neat cements and mortars mixed in the usual manner. These tests complete the group given in Table I. In all of our tests on neat cement we found considerable variation. This, we believe, is universally experienced and is hard to explain.

The data of Tables I and II lead us to the conclusion that *the strength of concrete is limited by the strength of the paste, and the amount of paste-strength developed in concrete is a function of the amount of paste per unit volume of concrete.*

EXCEPTIONS TO EQUATION 1

When we tried to apply Eq. 1 to data from other sources we at once ran into difficulties. We did not expect the constants given in Eq. 2 to hold universally, but it seemed reasonable that constants for Eq. 1 could be found which would fit any given series of tests. We tried the data given in Bulletin 137, University of Illinois Engineering Experiment Station, by Talbot and Richart, as this lent itself easily to our method of analysis. In their Table 7, Series 211, for example, where the relative water content and richness of the mix were variable, no equation could be found which was general for the whole table, nor would any water-cement ratio curve or voids-cement ratio curve do more than approximate the results. Fig. 4 illustrates the scatter from a curve of the form of Eq. 1. Fig. 29, page 65 of the Illinois bulletin shows the scatter from a voids-cement ratio curve, and Fig. 36 of the same bulletin shows the scatter from a water-cement ratio curve.

We noted also, in other tests, that the use of admixtures, notably diatomaceous earth, caused variation in strength not accounted for by

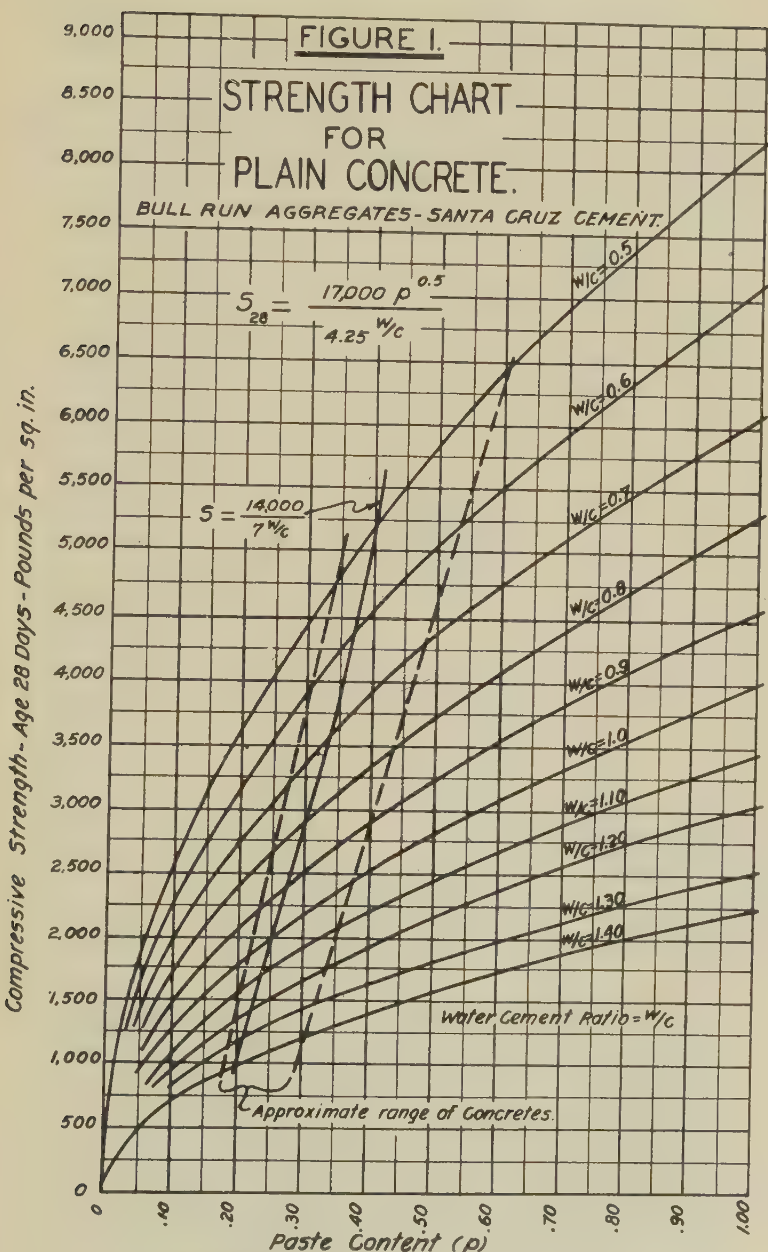
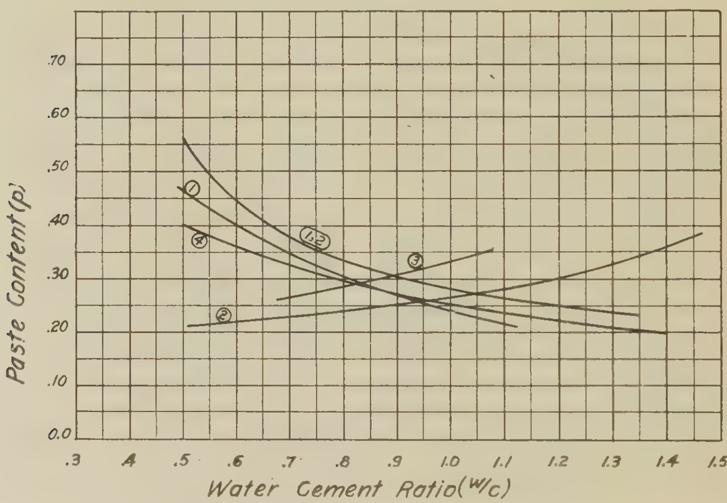


FIG. 1.—STRENGTH CHART FOR PLAIN CONCRETE.

any equation which would hold for the corresponding plain concrete. We began a study of the action of inert admixtures and finally summed up the following significant facts:

(1) We found that mortars containing diatomaceous earth were watertight at much higher water-cement ratios than the corresponding plain mortars. For example, a mortar made with sand having a fineness modulus of about 3.50 would leak visibly at water-ratios higher than 1.10. Addition of 8 per cent (of weight of cement) of diatomaceous earth raised the limiting water ratio to about 1.40.



① Richness of Mix Variable-Consistency & grading Constant-Method (1) of text.
(1.2) Richness of Mix and grading of aggregate Variable. Combination of Methods (1) and (2) of text.
② Variation of size and grading-Mix constant. Method (2) of text
③ Variation of relative consistency. Method (3) of text
④ Average from combination of methods.

FIG. 2.—RESULTS OF TESTS SHOWING UNIFORM BUT DIFFERENT RELATIONS BETWEEN w/c AND PASTE CONTENT, p.

Our permeability test specimens consisted of 6 x 12-in. cylinders containing a cylindrical muslin bag 2 x 8-in. filled with coarse sand. Water under pressure was applied to the sand bag by means of a pipe through the end of the cylinder to the end of the sand bag. Pressures up to 150 lb. per sq. in. were used. In testing, a metal cylinder form was fitted loosely around the specimen to act as a collector, and leakage dropped from this through a funnel to a glass receiver. Leakage around the pipe was kept separate from seepage through mortar. Humidity in the testing room was kept high to reduce evaporation.

For tests on concrete, we were able to include aggregate up to 3-in. by using nail kegs for forms and sand bags as in the 6 x 12-in. mortar specimens.

This method is, we believe, inferior to the method used by M. O. Withey and others, where the flow into the specimen, rather than leakage from the specimen, was measured, but it enabled us to determine certain limits with sufficient accuracy to draw some conclusions, especially since our tests confirmed in a general way other published tests on permeability.

We sought to determine the relation between permeability and water ratio and paste content for (1) mortars containing sand graded 0 to No. 4, fineness modulus about 3.50 with 2.3

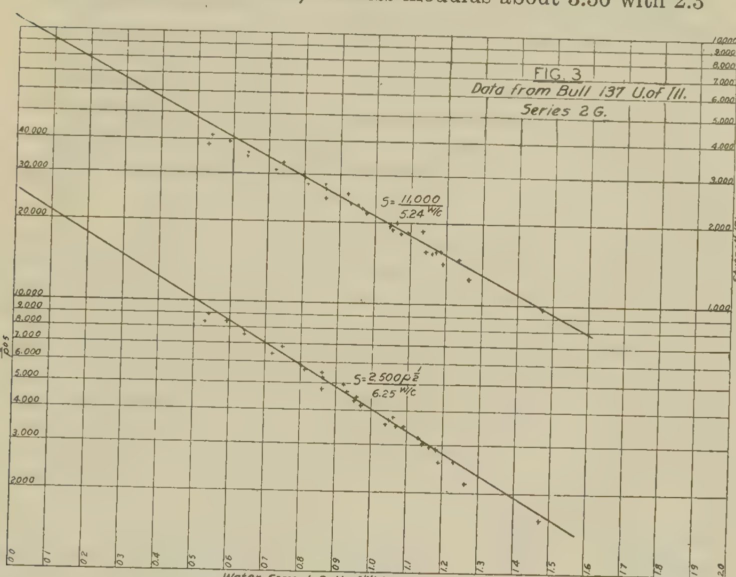


FIG. 3.—TAKEN FROM SERIES 2G. BULLETIN 137, U. OF ILL. ENGINEERING EXPERIMENT STATION.

per cent passing the 100-mesh sieve, (2) a parallel series containing sand graded 0 to No. 4, fineness modulus about 3.00 with 5.0 per cent passing the 100-mesh sieve, and (3) mixtures containing diatomaceous earth. For these tests curing conditions were kept constant. These tests together with information from other sources have led us to the following conclusions:

(*a*) For given materials and conditions of tests there is a very definite relation between water-cement ratio and permeability. Our tests on mortars showed that the limit for our coarse sand was about $w/c = 1.10$ for watertightness at 28 days. By watertightness we mean no observable leakage by the method described above.

(*b*) The presence of fine inert material has a marked effect on permeability. Using diatomaceous earth with the coarse sand, we found that 2 per cent (of weight of cement) made the mortar watertight at water ratios below 1.30. Adding 5 per cent, the limit was raised to some point between 1.2 and 1.56. The exact value was not determined. Eight per cent gave no leakage at $w/c = 1.33$.

Using the fine sand described above there was no leakage under 120 lb. per sq. in. at 60 days, water ratio 1.40. The specimens failed at 150 lb. per sq. in. No 28-day tests were secured. Incidentally, our tests showed that aging under good curing conditions reduced leakage, but if leakage was observable at 28 days it would not be absent at 60 days, so that we may assume that leakage of these 60-day specimens probably could not have been observable at 28 days.

(*c*) The actual numerical limits found in our tests are of little general value, for it can easily be seen that the characteristics of the fine aggregate may be an important factor. Determining of limits for permeability thus becomes a local problem.

(2) With given aggregate, the effectiveness of the admixture depended on the cement content. In lean mixes with high water ratios the beneficial effect was greatest. As the cement was increased and the water ratio lowered, the effect became less and less beneficial until at a water ratio of about 0.75 it had no effect on the strength. In richer mixes, giving water ratios less than 0.75, it caused a loss in strength. (See Fig. A.)

(3) The gain in strength and watertightness due to the use of diatomaceous earth is *not due to absorption* as is sometimes claimed. A direct test will show that the absorption is less than 1.0 per cent. Also, calculations of yield which include the volume of the water are not inaccurate when diatomaceous earth is used as they would be if any marked absorption took place; for absorption, to be effective in increasing the strength, must reduce the volume of the concrete (before the initial set) by the volume of the water absorbed. In most cases, if the admixture were to absorb the additional water required, so as to restore the original water-ratio, it *would have to absorb about twice its own absolute volume.*

(4) Gain in strength and watertightness over plain concrete from use of admixture may not be due to gain in density of the concrete or of the paste because the additional mixing water required is nearly always, if not always, greater than the absolute volume of the admixture. When the volume of concrete is increased by the addition of water, its density is decreased.

Incidentally, it might be well to emphasize the fact that the strength of concrete is not a function of its density. For example, Table I shows that neat cement having a water-cement ratio of 1.00 has a strength of about 4000 lb. at 28

days. The density of such a mixture is not over 0.46 including combined water. Put this paste in concrete and it develops but a fractional part of the 4000 lb., depending upon the amount of inert added. Ordinary concrete having a water-cement ratio of 1.00 has a paste content of about 0.25 and the strength is about 2000 lb. The density, however, rises to about 0.87. We have changed the density from 0.46 to 0.87 and reduced the strength to half its former value.

Again, ordinary concrete, having a water ratio of 0.6, paste content 0.40, has a strength of about 5000 lb. per sq. in. and a density of 0.86. The density is slightly less than the 2000-lb. concrete cited above. Bull Run concrete with a paste content of 0.20 and strength of 1800 lb. per sq. in. has a density of 0.89.

When the density of a given mixture is increased by reducing mixing water, however, the strength increases with the density. In a similar manner we have noticed that permeability is not dependent on density of the concrete. The size and distribution of voids seems to be more important than the total amount of voids.

Strength and permeability are dependent on the density of the cement paste, which is expressed as the water-cement ratio, and upon other factors brought out in this paper.

(5) It was noted that when lean concrete cylinders were broken transversely or in tension the aggregate in the plane of fracture always adhered to the upper half of the specimen. In rich mixes, and in mixes containing admixture, this tendency was much less or absent.

(6) Gain in strength with admixture could not be accounted for by gain in paste content due to the additional water because the rise in water-ratio would offset this effect many times over.

It seems, then, that more than the usual amount of fine material in the mix brings about a different relation between water ratio and strength, and between water ratio and permeability. Also, that this fine material in some way causes the aggregate to be more uniformly bonded. That is, we note that fine material decreases permeability without reducing voids, increases the strength without decreasing the water-cement ratio or increasing the paste content; and that when it is present the bottom surfaces of the aggregate particles tend to become bonded as firmly as the top surfaces.

Considering these facts we came to the conclusion that diatomaceous earth, or, we believe, any inert fine material, is beneficial to concrete in proportion to the degree to which it maintains the homogeneity of the cement paste, preventing the precipitation of part of the cement to the surfaces of the aggregate. When this precipitation or settling of the cement takes place, concentrating on the top surfaces of the stone and settling away from the lower surfaces, the concrete develops but a part of its possible strength; and it is less watertight because the cement is

no longer uniformly dispersed. That this settling away does take place is evidenced by the fact that the stones are more firmly bonded at their top surfaces, as described in item (5). Photographs published by Herbert J. Gilkey in the *Engineering News-Record* for February 10, 1927, well illustrate this effect. If the concentration of the cement in the water is sufficiently high, this precipitation does not take place, or, if the paste is "stiffened" by the addition of finer material, precipitation of the cement is prevented. That the benefit to watertightness is not small is shown by the difference in limiting values of water ratio cited above. The effect on strength is considerable. Plastic concrete having a water ratio of 1.25 may be stronger than semiplastic concrete having a water ratio of 1.10. The degree of plasticity reflects to some degree the condition of the cement paste.

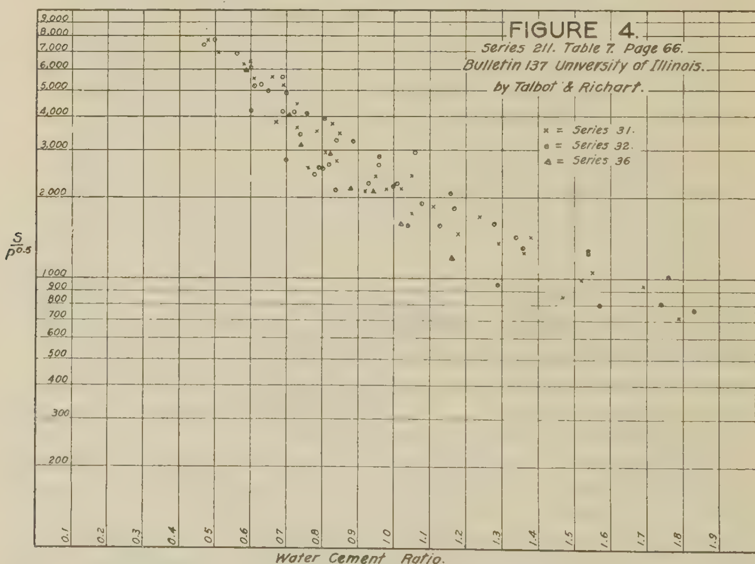


FIG. 4.—TAKEN FROM SERIES 211, TABLE 7, BULLETIN 137, U. OF ILL. ENGINEERING EXPERIMENTAL STATION.

On the other hand, an admixture is detrimental to the extent that it requires extra mixing water. At some point the beneficial plasticizing effect will be neutralized by the additional water required, and no change in strength will result, as shown by Fig. A. The position of this neutral point depends upon the original relation between cement, water, and aggregate.

THE THIRD VARIABLE

Referring again to Series 211 of Bulletin 137, University of Illinois Engineering Experiment Station, where we could find no general relation-

ship for the series, here the paste content and water ratio were varied by using different relative consistencies, or, relative water contents, so that for a given cement-aggregate ratio there would be several relative consistencies. We found that a curve which would approximate one relative consistency would not hold for another. Also, Table 4, taken from the international Critical Tables, Volume II, shows that when the water ratio is increased merely by increasing the water, the strength becomes relatively less. This is shown in the table by the calculated value of "B," col. 5, from Abrams' quation $S = \frac{14000}{B^{w/c}}$.

From this we conclude that changing the relative water content in this manner altered the physical condition of the paste by increasing the amount of cement precipitation, causing a variation in strength independent of the water-cement ratio or paste content. The use of an admixture, presence of clay, or any factor which effects the physical condition of the paste will alter the strength and permeability of the concrete. *Factors which effect the physical condition of the paste (its homogeneity) constitute the third variable effecting the potential strength of concrete.*

Summary

The results of this study may be summed up as follows:

(1) By the wet-screening method we find the strength of concrete to be variable at constant water-cement ratio.

(2) The premixed-paste method indicates that the variation is a function of the paste content. For our tests the relation is

$$S_{28} = \frac{17000p^{0.5}}{4.2^{w/c}}$$

(3) A study of the action of admixtures shows that the physical condition of the paste is a third variable.

(4) The potential strength of any plastic mixture of concrete is a function of:

(*a*) The concentration of cement in the water (w/c).
(*b*) The concentration of the paste in the concrete (p).
(*c*) The physical condition of the paste at the time of hardening, or, the amount of cement precipitation.

The significance of this last variable will be made more clear in the discussion which follows.

The Water-Cement Ratio Strength "Law"

If the foregoing is true, how is it to be reconciled with the much verified water-cement ratio "law" which not only seems to prove out in the laboratory but also works well in practice?

We believe the answer to this question will show the fallacy of

calling the strength-water-ratio relationship a law, but it will at the same time confirm its practicability as a working rule. While different investigators agree that the relation between S and w/c may be expressed by a single curve, the position of the curves do not well agree.[1] In Series 211 referred to above, the scatter from any single curve is too great to be considered accidental or the result of natural test variations. These differences have usually been ascribed to difference in materials and other "external" variables. These undoubtedly are contributing factors, but we believe there are "internal" variables which may easily cause these variations.

The three "internal" variables are the water-cement ratio, w/c, paste content, p, and the paste condition at the time of setting. Now, in running a series of tests with the purpose of determining the relationship of S to w/c the w/c may be varied by (1) changing the cement-aggregate ratio keeping consistency constant, (2) changing size and grading of the aggregate keeping the cement-aggregate ratio constant, (3) changing the relative consistency, or (4) by any combination of the foregoing.

In cases (1), (2), or (3) it can readily be seen that p would vary regularly with w/c, so that p may be considered to be a function of w/c, but it is important to note that the relation would be *different* in each case. (See Fig. 2.) In case (1) where w/c is dependent upon the richness of the mix, the relationship of p to w/c would be entirely different than in case (3); for example, where starting with a dry mix, w/c and p would be changed by adding more water. But whatever the p vs. w/c relationship, as long as there is a definite and uniform relationship, strength may be expressed as a function of w/c alone. It is obvious, however, that if the S vs. w/c relationship would be effected by the p vs. w/c relationship, with the same materials, the position of the water ratio curve would depend upon whether method (1), (2), or (3) had been used in the running of the series.

In other words, the water-ratio strength relationship depends upon what the relationship between water ratio and paste content is; and this relationship is dependent upon what method was used in running the series. Thus it is possible to obtain several different water-ratio strength curves for different series of combinations of the same materials.

The same reasoning holds for the third variable, the condition of the paste. For the sake of brevity, let us refer to this variable as the degree of plasticity since the plasticity of the mix reflects to some degree the condition of the paste.

In cases (1), (2), and (3), the degree of plasticity may also be a function of the water ratio, and in each case the relation is different. In case (1), starting with a rich mix and low w/c the plasticity usually decreases as the mix becomes leaner and w/c higher. In case (2) plasticity may be uniformly altered by uniformly altering the size and grading of

[1] See "A Method for Predicting Concrete Strengths with Increased Precision," by Herbert J. Gilkey, *Proceedings*, A.C.I. 1928, p. 155.

the aggregate. In case (3) starting with a plastic mixture, addition of water will increase precipitation of cement.

To sum up: the strength *vs.* w/c relation of a concrete test series made by any one of the first three methods outlined, (1) changing richness of mix, (2) changing size and grading of aggregate, or (3) changing relative consistency may be expressed by a single curve because of the three "internal" variables, (w/c, p, and plasticity), p and plasticity are functions of w/c. The position of the curve will depend upon which of the three methods has been used. (See Fig. 2.)

A fourth possibility is the combination of methods. Methods (1) and (2) may be combined so as to give a definite p vs. w/c relationship, and again the combination of methods may be such as to produce no uniform variation of p. The same is true of the plasticity variable. A combination of methods may produce results such that the strength relationship can not be expressed by a single curve, or even by two variables as in Eq. 2, because of the lack of relationship between the variables. It would take an equation in three variables to express this situation accurately. Such a case has previously been cited in Series 211 of *Bulletin* 137.

The scatter from a single curve will depend upon the manner and degree to which the methods are mixed. The original water-cement ratio curve given by Abrams in *Bulletin* 1, Structural Materials Research Laboratory, Lewis Institute, represents a mixture of methods, and a spread of 2000 lb. per sq. in. is not uncommon. This curve represents grand average concrete.

In our tests at Bull Run dam, outlined in the first part of this paper, there was no relation between p and w/c, hence no relation between S and w/c, but the plasticity varied in a uniform manner. An equation in two variables, as Eq. 2, was required to express the relationship. This equation is shown graphically in Fig. 1. Superimposed on the graph of Eq. 2 is the graph of $S = \dfrac{14000}{7^{w/c}}$, Abrams' original equation which, under proper conditions, holds very well for our materials. This illustrates that Abrams' equation could hold exactly only when the paste content has one certain value for each water ratio. For example, when $w/c = 1.00$, $p = 0.25$, and when $w/c = 0.5$, $p = 0.40$. Departure from these certain values causes scatter from the curve. The zone enclosed by the broken lines approximates the area of the chart covered by concrete ranging from ¾-in. maximum size aggregate to 6-in. maximum size.

In series 2G, *Bulletin* 137, method (2) was used; that is, w/c was altered by changing the size and grading of the aggregate, keeping the mix constant. Unlike method (1), this gives lowest paste content at lowest water ratios. The degree of plasticity would of necessity change in a uniform manner through the series. The relationship in this data

should therefore be expressible in either one, two, or three variables. Fig 3 shows that

$$S = \frac{11{,}000}{5.24^{w/c}} \qquad (3)$$

$$\text{or } S = \frac{25{,}000p^{0.5}}{6.25^{w/c}} \qquad (4)$$

hold about equally well, the average residual for (3) being 96.5 lb. and for (4) 90.0 lb.

The first equation gives 2100 lb. per sq. in. when $w/c = 1.00$. This is quite close to Abrams' original equation of $S = \frac{1400}{7^{w/c}}$ which gives 2000 lb. per sq. in. at $w/c = 1.00$. A nearly parallel curve giving 2100 lb. at $w/c = 1.00$ is $S = \frac{14000}{6.7^{w/c}}$, but the average residual is 200 lb. per sq. in. with a maximum of 1240 lb. Here, the relation of S to w/c in a series made by method (2) proved to be different than in "grand-average" concrete.

SUMMARY

(1) While the water-cement ratio "law" has been verified by many investigators it will not explain variations in the Bull Run tests, variations due to use of admixtures, or variations in position of the various w/c curves.

(2) The existence of a strength water-ratio relationship is contingent upon the existence of a uniform relationship between water-ratio, paste content and physical condition of the paste (homogeneity) which, for sake of brevity is spoken of as the degree of plasticity. This last named relationship determines the position of the water-ratio strength curve. Scatter from a single curve is due to lack of uniform relation between the three variables.

(3) When the strength of concrete can be expressed as a function of w/c it can also be expressed as a function of w/c and p, as shown by Series 2G, Fig. 3. But the reverse need not be true, as shown by the Bull Run tests (Table 1, and Fig. 1).

DISCUSSION

We anticipate certain objections to our Bull Run tests, the answers to which have not been included above.

(1) The time element may change the relationship.

Our tests included ages up to 3 mos. but there is not enough data for general conclusions, except that the strength differences are not reduced.

(2) The strength differences at constant w/c may be temporary due to acceleration caused by more heat-of-setting in the high-paste-content cylinders.

We embedded resistance thermometers in a set of cylinders made by the wet screening method and found that the highest paste content

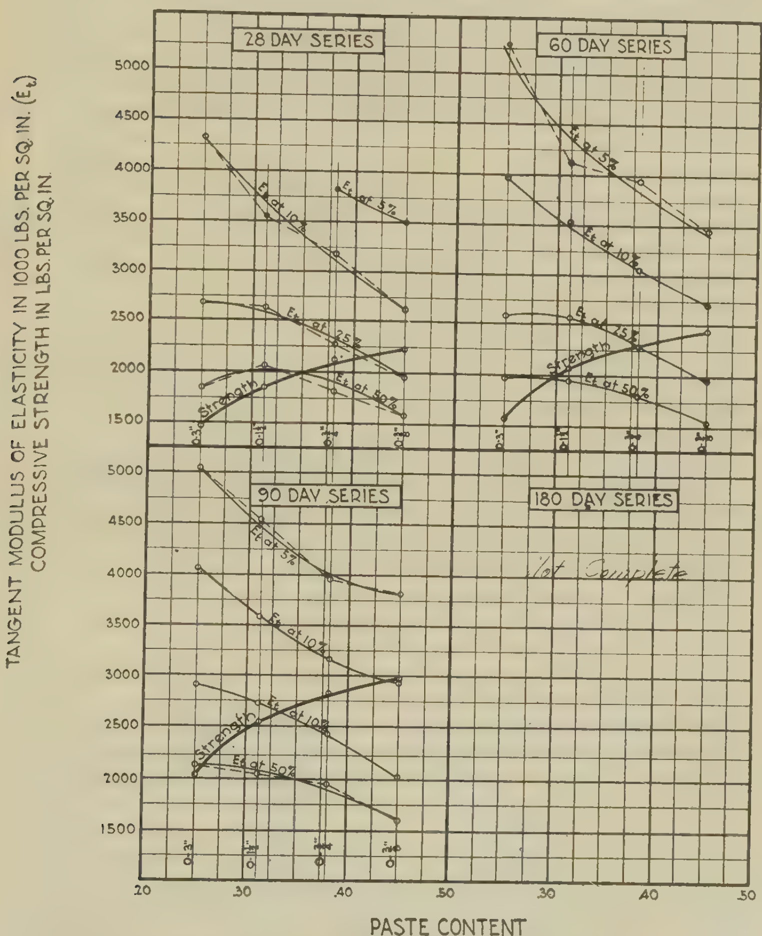


FIG. 5.—RELATION BETWEEN STRENGTH AND PASTE CONTENT AND TANGENT MODULUS AND PASTE CONTENT AT DIFFERENT LOADS.

reached a maximum 7 deg. C. higher than the lowest paste content. This difference in temperature would have to account for the high-paste-content cylinder being twice as strong as the low paste content cylinder, which seems impossible. Also, as noted above, the differences in strength

apparently do not decrease with time as they would if the cause were acceleration due to heat.

(3) The wet-screening method may change the water-ratio of the paste.

A large quantity of sand-cement grout was made up and divided into two portions. The first portion was stirred and turned continuously while coarse aggregate, just damp, was worked into the second portion. Then, three sets of cylinders were cast at the same time: set 1—original grout; set 2—grout screened out after having been mixed with coarse aggregate; and set 3—grout containing coarse aggregate with the following results:

Lab. Nos.	Set No.	w/c*	Strength, lb. per sq. in., 6 x 12-in. Cylinders 7 days	28 days
2070–2073........	1	1.10	1058	2208
2074–2077........	2	1.10	1043	2098
2066–2069........	3	1.10	787	1573

* Water loss assumed to be proportional to water content.

The data for set 3 was as follows: maximum size 1½in., fineness modulus 5.85, slump 2½ in. These indicate that screening does not alter the water ratio appreciably. The water ratio of sets 2 and 3 were probably raised a little by having the coarse aggregate too damp.

It should be mentioned that when specimens are made by the wet-screening method, or by the method just described, the consistency as measured by the slump cone varies greatly from one extreme of maximum size to the other. For example, portions of concrete screened from the same batch would show the following slumps:

0–3 in. aggregate–4½ in. slump
0–1½ " " –6½ " "
0– ¾ " " –8.0 " "
0– ⅜ " " –10.0 " "

But the actual consistency of the cement-water paste, or the grout (⅜-in maximum), would be judged to be the same in each case. Incidentally this emphasizes the necessity of careful sampling when making the slump test.

It was observed that water was lost from the molds during the process of making cylinders, and that more was lost from the higher paste contents. Gilkey[1] observed the same phenomena when studying this subject and reported that the water loss was proportional to the original cement and water content. We made this assumption in analyzing our data. We made no direct measurement on water losses, but we did measure the volume of the cylinders after they were hard. In Table V we have tabulated the results of measurement of sixty cylinders, 15 sets, made by the wet-screening method. The differences in volumes

[1] H. F. Gilkey: Aggregate as a Field for Needed Research, Discussion, p. 407, Vol. 23, A.C.I. *Proceedings*, 1927.

shown undoubtedly represent variation in molds, variation in the filling of the molds, and variation in skrinkage due to loss of water during the molding process, and perhaps shrinkage due to other causes not known.

However, taking the figures as they stand the loss is shown to be proportionally greater for the higher paste contents. The difference in water-cement ratio for the extremes is about 0.028, which would account for a variation in strength of about 6 to 7 per cent, while differences in strengths obtained are as high as 100 per cent of the lower strength.

Some of the results given in Table I were from mortars mixed in the usual manner and were just wet enough to be thoroughly plastic. The results from these were well predicted by results from the wet-screening method.

In general, slump or flow tests were not made in this series of tests because it was recognized that slumps of concretes having aggregate of different maximum sizes are not comparable. For example, the concrete containing aggregate graded 0–¾ in. is considerably drier than concrete graded 0–3 in. when they show equivalent slumps in the standard test.

Our object was always to use workable mixes, and cases where the mixes were not workable have been noted in the data.

(4) The strength may be altered by the ratio of the maximum size of aggregate to the diameter of the test cylinder so that in a large enough specimen, variation in strength at constant water-ratio might be different, or absent. This seems to us to be the strongest objection to our tests, for in the absence of direct tests on large specimens, it is not easily refuted.

We note that in changing from 0–¾-in. to 0–⅜-in. in our tests, the paste content changes about 10 per cent (from 30 to 40), and the strength is increased 170 lb. In the same test, changing from 0–3-in. to 0–1½-in. with 6 per cent increase in paste increased the strength 180 lb. Changing from 0–1½-in. to 0–¾-in., changed the paste content 10 per cent and raised the strength 250 lb. The shape of the curves in Fig. 1 indicate an increasing rate of reduction of strength with the reduction in paste content.

With this in mind, we believe that the use of a different size test specimen might result in a different constant for Eq. 2, but it is not likely that the strength variation could be eliminated or changed to any extent by the use of large specimens, for, in our tests, the variations persist even when the paste content is changed by changing from 0–⅜-in. to 0–No. 4 screen, or 0–No. 8 screen. Here the maximum particle size is quite small as compared with the diameter of the cylinder.

Such tests as we have seen reported on the subject of relation of maximum size to diameter of the cylinder are at variance but seem to indicate that rock up to half the diameter may be used without reducing the strength, and might tend to increase the strength due to intersection of shearing planes.

In some of our later tests we have noticed a tendency for the 0–3-in. test to fall below a smooth curve which fits the three smaller sizes screened out from the same batch.

(5) We wish to point out that the differences in paste content of concretes made with aggregates graded to different maximum sizes is exaggerated by the wet-screening method. Two mixes of the *same consistency*, one graded 0–¾-in. and the other graded 0–1½-in. would have less than 4 per cent difference in paste content at, say, water-ratio = 1.00, while the wet screening method would give about 10 per cent change in paste content. While we would find 300 lb. per sq. in. difference in the wet-

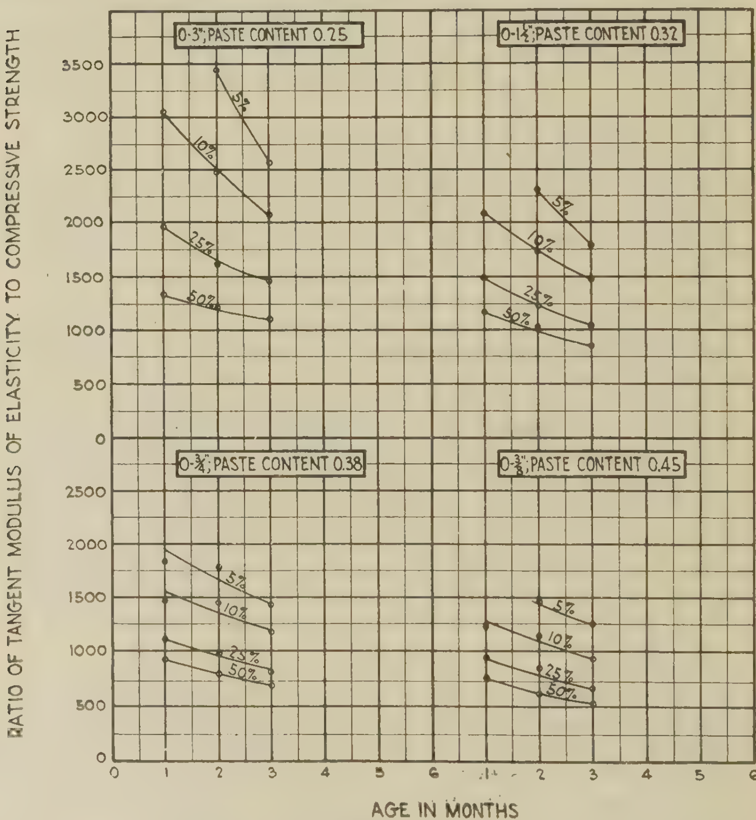


FIG. 6.—EFFECT OF AGE FOR DIFFERENT PASTE CONTENTS AT VARIOUS PERCENTAGES OF ULTIMATE LOAD.

screening method, in practice, using mixes of about the same consistency, the difference would be less than 150 lb. per sq. in. We believe that the effect of paste content can in most cases be ignored except where unusually large or unusually small aggregates are used. We are here implying, of course, that the size and grading of the aggregate governs the paste content at constant water-ratio.

(6) It should be mentioned that while this paper deals with but three variables, we are fully aware that there are many other factors that influence strength, some of them from the practical standpoint of more importance. Recognizing that the kind of cement and aggregate used are of importance, we deal with different combinations of the same materials. We refer to the "potential strength" of a mixture, thus eliminating from the discussion, time, temperature, personal equation, etc. For want of better terms, we speak of the three variables as "internal" variables. To our list of three might be added the "quality of materials" used. The actual strength obtained will depend upon the "external" variables (time, curing, temperature, etc.), which are largely factors of environment.

MODULUS OF ELASTICITY TESTS

A committee of the American Society of Civil Engineers is studying the deformation in the Bull Run dam by means of embedded telemeters,

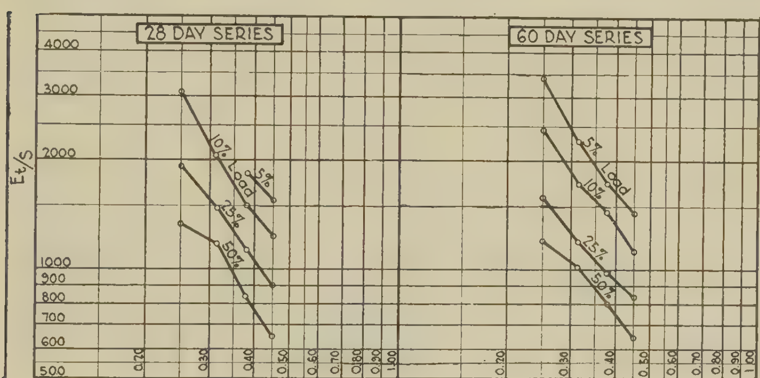


FIG. 7.—EFFECT OF PASTE CONTENT ON RATIO OF TANGENT MODULUS OF ELASTICITY TO STRENGTH.
For 28 and 60 day series.

resistance thermometers and the strain gauge. The essential data required includes the compressive strength and modulus of elasticity of the concrete.

Here again we were confronted with a problem similar to the one just discussed—"what is relation between the stress-strain relation in our test specimens, and the stress-strain relation of the whole mix?" If concrete may be thought of as being made up of two components, aggregate and paste, it would seem that the total deformation would be the resultant of the deformations in the aggregate and in the paste. This would make the paste content an important factor, and we would

expect the whole mix to be stiffer than the test portion because of the high modulus of elasticity of the aggregate. Tests were therefore designed to throw some light on this question.

Unfortunately, this problem was not in mind when we made most of our strength studies so we were obliged to repeat much of the work done before. The work is now under way but incomplete. In general, we follow the method of Stanton Walker[1] in obtaining the stress-strain curves, except that we are using the wet-screening method in making specimens.

Owing to the fact that this work had to be carried on along with our routine laboratory work, we could not make up closely controlled laboratory mixes for this series of tests. Instead, we took concrete from the large mixers on the job. The batches were the regular material being

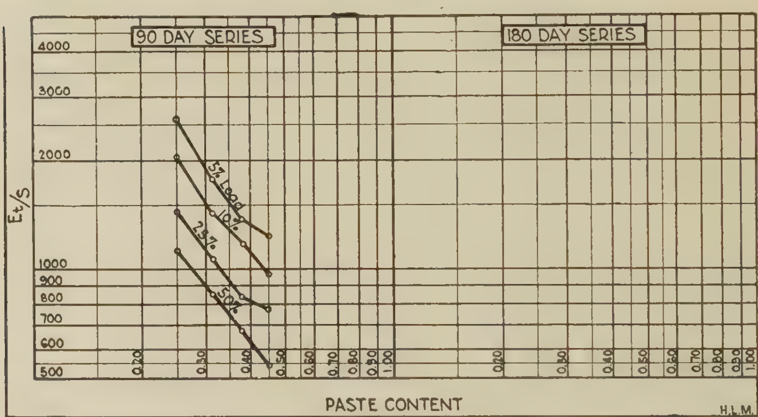


FIG. 8.—EFFECT OF PASTE CONTENT ON RATIO OF TANGENT MODULUS OF ELASTICITY TO STRENGTH.

For 90 day series; 180 day series not complete.

used in the dam, and included three different mixes designed to produce water-ratios of approximately 1.00, 1.10, and 1.20. In all, four series were cast, the grouping being made according to the age at test. The four series are: series 1, aged 28 days; series 2, aged 60 days; series 3, aged 90 days; series 4, aged 180 days. Each series comprised nine sets of four cylinders made by the wet-screening method, representing nine batches sampled on different days. Each series of nine included three sets each of the three classes of concrete.

Aggregate for the batches from which these cylinders were made (all of the batches for the job, in fact) was controlled and proportioned as follows: Bar-run material was washed, and separated by screens into bins containing approximately the following size: 0–⅜-in., ⅜-in.–1-in., 1-in.–2-in., 2-in.–5-in., and 5-in.–7-in. Sand was measured by inundators

[1] Modulus of Elasticity of Concrete, by Stanton Walker. *Bulletin 5*, Structural Materials Research Laboratory, Lewis Institute, Chicago.

and coarse aggregate by volumetric aggremeters in such proportions as to conform to a grading curve expressed by the following equation:

$$p = 1 - \left(\frac{d}{6}\right)^{0.45}$$

where p = per cent retained on sieve of size d
d = sieve size (clear opening in inches)

Cement was proportioned by bags, one bag being the smallest unit used. The batches were mixed in 2-yd. tilting mixers. The layout gave very close control over the quantity and grading of the coarse aggregate. Fluctuations in grading of the sand, however, gave considerable variation in the water requirements of the batch. Unfortunately, we found that the water measurement on the job was not precise enough to give us sufficiently accurate data for this series of tests. Without accurate knowledge of the water-content we could not compute the percentage composition of the batch and could not know the paste content or water-ratio exactly.

Without accurate knowledge of the water-ratio our only recourse was to determine the relative paste contents of the cylinders usually taking the 0–⅜-in. cylinder as unity. This can be done from the specific gravity of the cylinder and of the aggregate as follows:

$$a - c = \frac{b}{k}$$

where

a = Specific gravity of the aggregate
c = Specific gravity of concrete as measured before it has a chance to lose water by evaporation
k = A constant depending on the water-cement ratio
b = k times the paste content

The relative paste contents of a group of cylinders from the same batch are proportional to their respective values of b. To determine the absolute paste content it is necessary to know the water-cement ratio, or k.

By this method, we determined the average relative paste contents to be

0–3-in.	0.562
0–1½-in.	0.700
0–¾-in.	0.840
0–⅜-in.	1.00

The specific gravity of the aggregate was 2.72 and average specific gravity of the 0–⅜-in. cylinders was 2.24. Substituting these values in the equation given above b becomes 0.48. Assuming an average water-cement ratio of 1.10, the average absolute paste content of the 0–⅜-in. cylinders is 0.45. Computed by the relative paste contents

given above, the average absolute paste contents for each group of similar cylinders are:

0–3-in.	0.25
0–1½-in.	0.315
0–¾-in.	0.38
0–⅜-in.	0.45

Deformations are measured by an Ames dial and apparatus patterned after that used by Stanton Walker. A gauge length of 8 in. was used in all tests. Readings were taken at 100 lb. per sq. in. intervals, and with each reading the load was kept constant until the needle moved so slowly as to appear stationary. We find, as did Walker, that the stress-strain relation is of the form

$$s = kd^n$$

where s = stress
d = deformation
k = a constant
n = a constant exponent

The constants are effected by strength, paste content, and perhaps age.

Results of the tests now completed are tabulated in Tables VI, VII and VIII. To gain an idea of the trends the average values from each group are plotted in Figs. 5, 6, 7, and 8.

In Fig. 5 the relation between the tangent modulus of elasticity and paste content is plotted for different percentages of the ultimate load. On the same graph the trend of strength is shown. It is very evident that the paste content has a marked effect upon the relation between strength and modulus of elasticity. This is brought out in a different way in Figs. 7 and 8 where the ratio of the tangent modulus to strength is plotted against the paste content. Here the trend is unmistakable.

Fig. 6 shows that the ratio of modulus of elasticity to strength becomes less with age. In other words, the modulus of elasticity does not increase as rapidly as the strength. This confirms results obtained by Stanton Walker.

The fundamental lack of accuracy of our data precludes drawing definite conclusions. However, we feel that the trends are definite enough, and the differences due to paste-content variation of sufficient magnitude as to reveal the necessity for more well-organized research in this field. It is with the idea of pointing this out that we offer this data.

Acknowledgements—None of the work reported here would have been possible without the sympathy and support of Mr. Morrow, Mr. Torpen, and D. C. Henny, consulting hydraulic engineer, all of whom have been a constant source of help and encouragement. Henry Mason, laboratory technician, rendered invaluable aid in carrying out the tests.

TABLE 1—EFFECT OF PASTE CONTENT—BULL RUN TESTS
Specimens aged 28 days in damp sand, tested damp

w/c	Paste Content p	Size of Aggregate	Test Result	$S=\frac{1700p^{0.5}}{4.2^{w/c}}$	Residual	$S=\frac{14000}{7^{w/c}}$	Residual
0.45	1.00	....	9044	8900	144	5800	3244
0.50	1.00	....	8426	8350	76	5300	3126
0.50	0.96	0–No. 14	7505	8100	595	5300	2205
0.525	1.00	....	8480	8000	480	5000	3480
0.60	1.00	....	6878	7200	322	4350	2528
0.635	1.00	....	6963	6800	163	4050	2913
*0.75	0.30	0–3 in.	3240	3180	60	3750	510
*0.75	0.72	0–3/8 in.	4850	4900	50	3750	1100
0.80	0.70	0–No. 8	4700	4500	250	2950	1900
0.80	0.945	Diatomite	4655	5240	585	2950	1705
1.00	0.94	Diatomite	4200	4000	200	2000	2200
1.00	0.25	0–2 in.	2000	2000	000	2000	0000
1.10	0.927	Diatomite	3200	3340	140	1650	1550
*1.13	0.275	0–3 in.	1785	1785	000	1550	235
*1.13	0.340	0–1½ in.	1820	1980	160	1550	270
*1.13	0.410	0–¾ in.	2190	2175	15	1550	640
*1.13	0.500	0–3/8 in.	2420	2400	20	1550	870
*1.20	0.20	0–3 in.	1270	1380	110	1360	90
*1.20	0.25	0–1½ in.	1710	1540	160	1360	350
*1.20	0.30	0–¾ in.	1680	1690	10	1360	320
*1.20	0.41	0–3/8 in.	1840	1970	130	1360	480
*1.20	0.49	0–No. 8	2000	2150	150	1360	640
*1.20	0.68	0–No. 8	2580	2540	40	1360	1220
1.20	0.61	0–No. 8	2423	2410	13	1360	1063
*1.20	0.912	Diatomite	2750	2940	190	1360	1390
1.40	0.910	Diatomite	1900	2150	250	915	985
Average...	....	....	....	....	165	....	1350

* Wet-screening method.

TABLE 2—EFFECT OF PASTE CONTENT BY PREMIXED-PASTE METHOD
Age at Test, 2 Days

w/c	p	Size of Aggregate	Test Result	$\frac{S}{p^{0.5}}$
INCREASING MAXIMUM SIZE; w/c CONSTANT				
0.635	1.00	....	1870	1870
0.635	0.90	No. 100–No. 28	1875	1980
0.635	0.80	No. 100–No. 8	1770	1980
0.635	0.74	No. 100–No. 4	1710	1980
0.635	0.55	0–¾ in.	1360	1830
0.635	0.35	0–2 in.	1095	1840
Average......	....	....	....	1913
MAXIMUM SIZE CONSTANT AT ¾ IN.; MIX VARIABLE; w/c CONSTANT				
0.635	1.00	....	1870	1870
0.635	0.90	0–¾ in.	1840	1930
0.635	0.68	0–¾ in.	1450	1770
0.635	0.50	0–¾ in.	1310	1850
0.635	0.35	0–¾ in.	*1010	....
0.635	0.80	0–¾ in.	1730	1930
Average......	....	....	....	1870
SINGLE-SIZE AGGREGATES, MAXIMUM SIZE VARIABLE; MIX VARIABLE; w/c CONSTANT				
0.635	1.00	....	1870	1870
0.635	0.90	No. 100–No. 28	1875	1980
0.635	0.60	No. 8–No. 4	1505	1940
0.635	0.60	No. 4–⅜ in.	1505	1940
0.635	0.50	⅜ in.–¾ in.	*795	....
0.635	0.50	1½ in.–2 in.	*795	....
Average......	....	....	....	1942

SUMMARY

Increasing Maximum Size	1913
Maximum Size Constant..	1870
Single Size..	1942
Average	1908

* Not plastic.

TABLE 3—EXPERIMENTAL VALUES FROM SERIES 2G, UNIVERSITY OF ILLINOIS, BULLETIN 137

Cylinder No.	w/c	p	Obser. S_{28}	$S_{28} = \frac{25000\ p^{0.5}}{6.25^{w/c}}$	Residual	$S = \frac{11000}{5.24^{w/c}}$	Residual	$S = \frac{14000}{6.7^{w/c}}$	Residual
21	1.47	36.0	980	1025	45	980	0	1030	50
24	1.26	32.0	1240	1420	180	1350	110	1300	60
44	1.23	31.0	1490	1480	10	1450	40	1360	130
57	1.19	29.5	1410	1520	110	1550	140	1460	50
92	1.18	29.5	1570	1580	10	1560	10	1500	70
71	1.17	29.0	1570	1590	20	1590	20	1520	50
106	1.16	29.0	1550	1620	70	1610	60	1550	0
45	1.14	29.0	1680	1690	10	1670	10	1620	60
58	1.13	29.0	1870	1730	140	1700	170	1640	230
72	1.09	28.0	1850	1800	50	1800	50	1780	70
93	1.07	27.5	1840	1850	10	1870	30	1840	0
59	1.06	27.5	2000	1900	100	1900	100	1890	110
107	1.05	26.5	1900	1910	10	1920	20	1900	0
46	1.04	29.5	1940	2040	100	1960	20	1950	10
94	0.97	25.5	2160	2170	10	2200	40	2220	60
73	0.96	25.0	2270	2180	90	2220	50	2280	10
47	0.95	28.0	2320	2330	10	2270	50	2320	0
108	0.93	25.2	2350	2310	40	2350	0	2400	50
60	0.92	25.5	2530	2370	160	2400	130	2460	130
49	0.86	27.4	2490	2750	260	2650	160	2750	260
95	0.86	24.8	2650	2620	30	2650	0	2750	100
74	0.86	24.6	2740	2610	130	2650	90	2750	10
109	0.81	23.0	2720	2740	20	2850	130	3000	280
61	0.80	24.0	2930	2850	80	2900	30	3070	140
75	0.74	23.0	3320	3130	190	3200	120	3450	130
96	0.72	23.0	3110	3220	110	3300	190	3600	490
76	0.64	22.5	3670	3720	50	3750	80	3320	350
110	0.64	22.0	3570	3680	110	3750	80	4200	630
97	0.59	21.5	3960	3950	10	4100	140	4600	640
98	0.54	21.0	4170	4280	110	4450	280	5000	830
111	0.53	21.0	3860	4370	510	4500	640	5100	1240
Average..	....	....	....	....	90	....	96.5	....	201

TABLE 4—EFFECT OF ADDING WATER TO CONCRETE
INTERNATIONAL CRITICAL TABLES. VOL. II.

Water Ratio	Paste Content	Relative Consistency	Compressive Strength	B*
MIX, BY VOLUME 1:4. MAXIMUM SIZE 1½ IN. FINENESS MODULUS 5.65				
0.68	0.277	0.90	3760	6.92
0.72	0.283	0.95	3280	7.50
0.75	0.288	1.00	3100	7.45
0.78	0.294	1.05	2720	8.13
0.82	0.300	1.10	2580	7.85
0.92	0.316	1.25	1920	8.67
1 08	0.349	1.50	1300	9.06

* Value of B of $S = \dfrac{14000.}{B^{w/c}}$

TABLE 5—COMPARISON OF AMOUNTS OF WATER LOST FROM CYLINDERS MADE BY THE WET-SCREENING METHOD

NOTE: These volumes were measured on the days the cylinders were broken, ages varying from 7 days to 6 months. All specimens were kept in damp sand up until the day to be tested, so they may be considered to be saturated. Otherwise volume changes due to moisture change would have to be taken into account

Laboratory Numbers	Volume of Cylinders in Cu. Ft. Determined by Water Displacement			
	0–⅜ in.	0–¾ in.	0–1½ in.	0–3 in.
427–31	0.198	0.197	0.193	0.198
761–67	0.192	0.195	0.197	0.196
785–88	0.195	0.195	0.198	0.198
616–19	0 193	0.195	0.198	0.197
625–28	0.196	0.197	0.196	0.199
631–34	0 196	0.197	0.199	0.195
2535-8	0 200	0.198	0.198	0.197
3450-3	0.194	0.198	0.198	0.199
3536-9	0 196	0.195	0.200	0.198
2772-5	0.196	0.196	0.198	0.198
3557-6	0.197	0.198	0.198	0.196
3631-4	0.196	0.196	0.198	0.200
2501-4	0.195	0.196	0.195	0.198
2221-5	0.199	0.199	0.199	0.199
2182-5	0.196	0.198	0.198	0.199
Total	2.939	2.951	2.964	2.967
Average	0.196	0.1967	0.1976	0.1978
Original volume assumed as 0.1980; loss	0.0020	0.0013	0.0004	0.0002
Loss per cu. ft. of concrete	0.0102	0.0066	0.0020	0.0010
Assuming original w/c = 1.00; w/c after loss	0.966	0.974	0.991	0.994

TABLE 6—MODULUS OF ELASTICITY TESTS

28-Day Series

WET-SCREENING METHOD

Laboratory No.	Strength	n	k in 1000 lb. per sq. in.	Secant Modulus in 1000 lb. per sq. in. for Different Percentages of Ultimate Strength and E/Strength								Tangent Modulus in 1000 lb. per sq. in. for Different Percentages of Ultimate Strength and E/Strength							
				E 5%	E/S 5%	E 10%	E/S 10%	E 25%	E/S 25%	E 50%	E/S 50%	E_t 5%	E/S 5%	E_t 10%	E/S 10%	E_t 25%	E/S 25%	E_t 50%	E/S 50%
AGGREGATE 0–3 IN.; AVERAGE PASTE CONTENT 0.25																			
2717.......	1235	0.602	89.6	....	....	6870	5570	3850	3120	2380	1930	....	....	4150	3360	2320	1880	1440	1168
2730.......	1220	0.557	55.5	....	....	11700	9600	5450	4470	2170	1780	....	....	6460	5300	3000	2460	1200	985
3569.......	1120	0.740	301.0	....	....	4870	4350	3500	3130	2700	2410	....	....	3620	3230	2600	2320	2000	1790
2739.......	2035	0.711	324.0	9100	4470	6550	3220	4440	2180	3420	1680	6490	3180	4690	2300	3170	1560	2440	1200
3579.......	1390	0.728	283.0	6350	4570	5060	3640	3550	2560	2780	3000	4630	3380	3700	2660	2600	1870	2030	1460
3651.......	1460	0.717	288.0	....	....	5850	4000	4060	2780	3040	2080	....	....	4200	2880	2920	2000	2180	1490
3604.......	1160	0.748	224.0	5800	5000	4650	4010	3450	2980	2830	2440	4350	3740	3490	3010	2590	2230	2120	1830
3771.......	1510	0.575	80.0	....	....	6560	4350	4220	2730	2020	2000	....	....	3800	2520	2430	1610	1740	1150
3755.......	1940	0.603	112.0	11400	5870	7300	3760	4050	2090	2550	1310	6900	3560	4420	2280	2450	1260	1540	795
Average....	1451	0.664	206.4	....	....	6500	4722	4070	2900	2765	2070	....	....	4380	3060	2680	1910	1850	1320
AGGREGATE 0–1½-IN.; AVERAGE PASTE CONTENT 0.315																			
2718.......	2105	0.833	72.5	4400	2090	3750	1780	3080	1460	2800	1330	3680	1750	3140	1490	2580	1230	2340	1110
2731.......	1535	0.694	221.0	....	....	5300	3460	3650	2380	2650	1730	....	....	3690	2410	2540	1650	1840	1200
3570.......	1450	0.778	426.0	....	....	3720	2560	3150	2170	2650	1830	....	....	2900	3000	2460	1700	2070	1430
2740.......	2580	0.690	267.0	8700	3370	6220	2410	4050	1570	3000	1160	6020	2330	4300	1680	2800	1090	2070	805
3580.......	1280	0.739	325.0	....	....	5340	4170	3800	2970	2970	2320	....	....	3960	3100	2810	2190	2200	1720
3652.......	1810	0.766	402.0	5300	2930	4200	2320	3240	1790	2600	1440	4070	2250	3230	1790	2490	1380	2000	1105
3605.......	1740	0.748	368.0	....	....	4840	2780	3480	2000	2790	1605	....	....	3630	2090	2620	1510	2100	1210
3772.......	2170	0.778	523.0	5850	2700	4800	2210	3920	1810	3100	1430	4560	2140	3740	1720	3060	1410	2420	1110
3756.......	2190	0.744	355.0	5500	2510	4460	2040	3320	1510	2680	1220	4100	1870	3320	1515	2460	1120	2000	913
Average....	1873	0.750	402.4	....	....	4740	2640	3540	1970	2800	1560	....	....	3550	2090	2640	1475	2120	1180

TABLE 6—CONTINUED

Laboratory No.	Strength	n	k in 1000 lb. per sq. in.	Secant Modulus in 100-lb. per sq. in. for Different Percentages of Ultimate Strength, and E/Strength: E 5%	E/S 5%	E 10%	E/S 10%	E 25%	E/S 25%	E 50%	E/ 50%	Tangent Modulus in 1000 lb. per sq. in. for Different Percentage of Ultimate Strength, and E/Strength: Et 5%	E/S 5%	Et 10%	E/S 10%	Et 25%	E/S 25%	Et 50%	E/S 50%
AGGREGATE 0–¾-IN.; AVERAGE PASTE CONTENT 0.38																			
2719.......	2000	0.728	278.0	5560	2780	4200	2100	2940	1470	2280	1140	4060	2030	3060	1530	2860	1430	1660	830
2732.......	1660	0.718	228.0	4900	2950	3860	2320	2680	1620	2080	1250	3520	2120	2770	1670	1925	1160	1495	900
3571.......	1560	0.746	265.0	....	....	3320	2130	2440	1560	1950	1250	....	....	2480	1590	1825	1170	1460	935
2741.......	2950	0.690	253.0	7350	2490	5290	1800	3500	1190	2600	880	5080	1720	3650	1240	2420	785	1800	610
3581.......	1740	0.799	386.0	4100	2360	3420	1970	2640	1520	2180	1250	3280	1890	2730	1570	2110	1220	1970	1130
3653.......	1990	0.805	540.0	4350	2190	3700	1860	2980	1500	2460	1230	3500	1760	2980	1500	2400	1200	1990	1000
3606.......	1970	0.773	408.0	4700	2390	3860	1960	2950	1500	2440	1240	3640	1850	2990	1520	2280	1160	1890	963
3773.......	2050	0.800	546.0	4750	2310	3950	1920	3100	1510	2780	1360	3800	1850	3160	1550	2480	1210	2230	1090
3757.......	2480	0.694	224.0	5400	2180	4350	1750	3170	1275	2350	950	3750	1510	3020	1210	2200	885	1630	655
Average....	2156	0.75	326.4	5140	2456	3883	1980	2933	1460	2340	1175	3840	1840	3200	1490	2280	1135	1800	903
AGGREGATE 0–⅜-IN.; AVERAGE PASTE CONTENT 0.45																			
2720.......	2240	0.686	173 0	5100	2280	3680	1650	2440	1090	....	....	3500	1570	2530	1130	1660	745	....	....
2733.......	1855	0.687	179.0	5500	2960	4040	2180	2650	1425	1960	1055	3780	2040	2780	1500	1830	985	1340	730
3572.......	1820	0.808	408.0	....	....	2620	1620	2120	1310	1800	1110	....	....	2120	1310	1720	1060	1460	900
2742.......	3000	0.767	426.0	5000	1670	3850	1280	2950	985	2420	810	3850	1280	2960	990	2270	758	1860	620
3582.......	1870	0 753	298.0	4450	2380	3350	1790	2470	1320	1950	1040	3350	1790	2530	1350	1860	995	1470	785
3654.......	2030	0.730	263.0	4750	2340	3700	1820	2600	1280	2040	1005	3480	1710	2710	1330	1900	935	1500	736
3607.......	2010	0.735	279.0	4750	2360	3760	1870	2650	1320	3320	1150	3500	1740	2770	1375	1950	970	1710	850
3774.......	2740	0.820	628.0	3920	1430	3380	1240	2760	1005	2410	880	3220	1170	2780	1015	2270	827	1980	722
3758.......	2610	0.748	333.0	4650	1780	3720	1425	2710	1040	2170	834	3480	1340	2790	1070	2030	780	1625	623
Average....	2220	0.748	332.0	4765	2150	3567	1653	2600	1197	2130	985	3500	1580	2660	1230	1940	895	1630	745

TABLE 7—MODULUS OF ELASTICITY TESTS

60-Day Series

WET-SCREENING METHOD

Laboratory No.	Strength	n	k in 1000 lb. per sq. in.	Secant Modulus in 1000 lb. per sq. in. for Different Percentages of Ultimate Strength and E/Strength								Tangent Modulus in 1000 lb. per sq. in. for Different Percentages of Ultimate Strength and E/Strength							
				E 5%	E/S 5%	E 10%	E/S 10%	E 25%	E/S 25%	E 50%	E/S 50%	E_t 5%	E/S 5%	E_t 10%	E/S 10%	E_t 25%	E/S 25%	E_t 50%	E/S 50%
AGGREGATE 0-3-IN.; AVERAGE PASTE CONTENT 0.25																			
2017.......	1200	0.625	95.0	8100	6750	5340	4450	3060	2550	2000	1670	5060	4220	3340	2780	1910	1590	1250	1040
2312.......	1100	0.630	106.0	9100	8270	6040	5490	3480	3160	2310	2100	5740	5320	3800	3460	2190	1990	1680	1530
2804.......	1800	0.706	254.0	7080	3930	5300	2940	3570	2040	2670	1480	5000	2780	3740	2140	2520	1400	1660	922
1994.......	1910	0.602	105.0	10470	5480	6580	3450	3680	1930	2350	1230	6300	3300	3940	2060	2220	1160	1805	946
2037.......	1730	0.607	121.0	12700	7340	8310	4810	4710	2720	3000	1730	7710	4460	5050	2920	2860	1710	1820	1050
3507.......	2350	0.802	636.0	5400	2300	4550	1900	3530	1510	3060	1300	4330	1850	3650	1560	2830	1210	2460	1050
3666.......	2080	0.756	439.0	6500	3120	5200	2500	4000	1925	3060	1470	4920	2360	3930	1885	3020	1450	2320	1115
3692.......	1440	0.785	572.0	6370	4430	5470	3800	4280	2980	3600	2500	5000	3480	4300	2990	3360	2340	2820	1960
3728.......	1610	0.729	380.0	6430	3950	4950	3080	3540	2200	2730	1700	4680	2910	3610	2240	2580	1610	1990	1240
Average....	1691	0.694	293.0	8050	4970	5720	3600	3780	2340	2750	1690	5300	3400	3930	2450	2610	1610	1980	1200
AGGREGATE 0-1½-IN.; AVERAGE PASTE CONTENT 0.315																			
2313.......	1375	0.656	138.0	7170	5220	5190	3780	3180	2310	2260	1650	4700	3430	3400	2480	2085	1520	1480	1075
2805.......	1850	0.778	425.0	4670	2530	3860	2090	2990	1620	2470	1340	3640	1970	3000	1620	2330	1260	1920	1040
1995.......	2070	0.622	118.0	7400	3570	5300	2560	3410	1640	2400	1160	4600	2210	3300	1590	2120	1020	1490	720
2038.......	2090	0.620	136.0	11000	5270	7330	3510	4080	1950	2045	980	6810	3260	4540	2170	2530	1210	1520	730
3508.......	2670	0.789	572.0	6250	2350	4570	1720	3530	1320	2900	1090	4930	1850	3610	1350	2780	1045	2290	860
3667.......	2320	0.775	503.0	5800	2500	4730	2040	4000	1725	2980	1285	4490	1940	3660	1570	3100	1330	2310	995
3693.......	2440	0.787	578.0	5540	2270	4520	1860	3330	1365	2900	1190	4360	1775	3560	1460	2620	1070	2280	935
3729.......	2130	0.823	673.0	4430	2080	3800	1780	3220	1510	2700	1270	3640	1710	3120	1460	2570	1210	2220	1040
Average....	2118	0.731	393.0	6550	3220	4900	2420	3460	1680	2580	1260	4150	2280	3520	1710	2520	1200	1930	1030

TABLE 7—CONTINUED

Laboratory No.	Strength	n	k in 1000 lb. per sq. in.	Secant Modulus in 100-lb. per sq. in. for Different Percentages of Ultimate Strength, and E/Strength								Tangent Modulus in 1000 lb. per sq. in. for Different Percentage of Ultimate Strength, and E/Strength							
				E 5%	E/S 5%	E 10%	E/S 10%	E 25%	E/S 25%	E 50%	E/S 50%	E*t* 5%	E/S 5%	E*t* 10%	E/S 10%	E*t* 25%	E/S 25%	E*t* 50%	E/S 50%
AGGREGATE 0-¾-IN.; AVERAGE PASTE CONTENT 0.38																			
2019	1610	0.674	158.0	6180	3840	4350	2700	2800	1740	2500	1550	4170	2590	2930	2440	1890	1170	1690	1050
2314	1540	0.720	189.0	3420	2220	3050	1980	2120	1380	1635	1060	2460	1660	2200	1430	1525	990	1170	760
2806	1860	0.750	316.0	4580	2460	3720	2000	2770	1490	2240	1200	3450	1850	2800	1500	2080	1120	1690	910
1996	2315	0.735	301.0	5150	2220	3960	1700	2880	1240	2240	965	3800	1630	2910	1250	2120	910	1650	710
2039	2560	0.758	406.0	5125	2000	4120	1610	3130	1220	2560	1000	3900	1520	3120	1220	2380	930	1940	760
3509	2840	0.778	499.0	5070	1790	4100	1475	3160	1110	2620	925	3950	1390	3200	1130	2460	870	2040	720
3668	2790	0.798	582.0	4820	1730	4050	1450	3200	1150	2730	1015	3850	1380	3240	1150	2560	920	2180	780
3694	2650	0.699	286.0	7700	2900	5760	2180	3840	1450	2880	1090	5460	2060	3970	1500	2700	1020	2020	764
3730	2390	0.725	302 0	5970	2500	4500	1890	3180	1330	2460	1030	4320	1810	3260	1365	2300	965	1780	745
Average	2295	0.738	338.0	5350	2410	4180	1890	3010	1340	2430	1090	3940	1770	3070	1450	2230	990	1800	800
AGGREGATE 0-⅜-IN.; AVERAGE PASTE CONTENT 0.45																			
2020	1750	0.737	231.0	3800	2170	3020	1730	2100	1200	1630	930	2800	1600	2230	1270	1550	890	1200	685
2315	1550	0.742	219.0	3400	3190	2740	1760	2000	1290	1570	1010	2520	1625	2030	1310	1485	960	1160	750
2807	1990	0.716	228.0	4960	2500	3830	1920	2650	1330	1990	1000	3570	1800	2750	1380	1900	955	1430	720
1997	2420	0.710	225.0	4840	2000	3670	1520	2550	1055	1950	810	3440	1420	2610	1080	1810	750	1390	575
2040	2740	0 765	407.0	4820	1760	3860	1410	2880	1050	2320	850	3690	1340	2950	1080	2200	805	1775	650
3510	3170	0.746	363.0	5200	1640	4070	1280	2960	935	2400	760	3890	1225	3040	976	2210	700	1790	565
3667	3120	0.724	323.0	5910	1900	4450	1430	3160	1010	2440	785	4280	1370	3220	1030	2290	735	1770	568
3695	2530	0.796	495.0	4140	1640	3420	1350	2670	1060	2260	895	3200	1310	2740	1085	2130	885	1800	710
3731	2670	0.772	410.0	4450	1665	3610	1350	2720	1020	2180	820	3450	1290	2800	1050	2100	790	1690	635
Average	2438	0.745	322 0	4620	1950	3670	1535	2640	1100	2080	875	3430	1445	2710	1140	1970	830	1560	650

TABLE 8—MODULUS OF ELASTICITY TESTS

90-Day Series

WET-SCREENING METHOD

Laboratory No.	Strength	n	k in 1000 lb. per sq. in.	Secant Modulus in 1000 lb. per sq. in. for Different Percentages of Ultimate Strength and E/Strength								Tangent Modulus in 1000 lb. per sq. in. for Different Percentages of Ultimate Strength and E/Strength							
				E 5%	E/S 5%	E 10%	E/S 10%	E 25%	E/S 25%	E 50%	E/S 50%	E_t 5%	E/S 5%	E_t 10%	E/S 10%	E_t 25%	E/S 25%	E_t 50%	E/S 50%
AGGREGATE 0–3-IN.; AVERAGE PASTE CONTENT 0.25																			
2278	1630	0.679	174.0	6680	4100	4820	2970	3140	1930	2290	1410	4530	2780	3270	2010	2130	1310	1560	960
2501	1950	0.721	368.0	7800	4000	5980	3070	4180	2140	3600	1850	5620	2880	4310	2210	3020	1550	2590	1330
2535	2000	0.727	336.0	7130	3570	5430	2720	3840	1925	3000	1500	5180	2600	3940	1975	2790	1400	2180	1095
2445	2220	0.788	573.0	5540	2500	4620	2030	3580	1620	3040	1470	4360	1970	3640	1640	2820	1270	2400	1080
3450	2030	0.696	249.0	7700	3800	5710	2810	3760	1850	2740	1350	5350	2640	3970	1960	2620	1290	1910	940
3536*	1810	0.548	78.7	20900	17000	11600	6400	6200	3420	2720	2050	16930	9350	6350	3500	3400	1870	1710	945
2772	1900	0.810	781.0	6460	3400	5580	2940	4480	2360	3880	2050	5230	2750	4530	2390	3640	1920	3140	1650
3557	2410	0.688	274.0	8850	3680	6600	2740	4370	1820	3210	1330	6090	2530	4540	1880	3010	1250	2210	920
3631*	2200	0.566	101.0	19200	8750	11200	5100	5500	2500	3230	1470	10870	4960	6340	2900	3110	1420	1830	835
Average of 7	2020	0.730	394.0	7160	3580	5400	2760	3910	1950	3110	1570	5050	2600	4030	2070	2860	1430	2280	1140
Average of 9	2050	0.692	326.0	11100	5650	6850	3430	4390	2180	3130	1610	6020	3610	4550	2270	2950	1480	2170	1080
AGGREGATE 0–1½-IN.; AVERAGE PASTE CONTENT 0.315																			
2279	1820	0.739	280.0	4810	2650	3760	2070	2720	1500	2140	1180	3560	1960	2780	1540	2010	1110	1580	870
2502	2550	0.765	528.0	6790	2660	5430	2120	4090	1600	3310	1300	5190	2030	4150	1630	3130	1225	2530	992
2536	2400	0.759	413.0	5460	2280	4370	1820	3290	1370	2640	1100	4140	1730	3320	1380	2500	1040	2000	835
2446	2840	0.849	933.0	4500	1590	3950	1390	3400	1200	2960	1040	3820	1340	3360	1180	2880	1015	2510	885
3451	2740	0.805	740.0	5950	2180	4980	1820	3980	1460	3380	1240	4780	1750	4020	1470	3200	1170	2720	994
3537	2210	0.799	582.0	5020	2270	4210	1910	3330	1510	2790	1260	4020	1820	3360	1520	2660	1200	2230	1010
2773	2930	0.833	881.0	5050	1730	4340	1490	3570	1225	3220	1100	4210	1430	3620	1240	2980	1020	2680	920
3558*	2620	0.514	53.3	9700	3700	7700	3950	4450	1700	2670	1020	4990	1910	3960	1520	2290	880	1370	530
3632*	3020	0.652	209.0	9900	3280	6140	2040	4180	1390	2900	965	6460	2140	4010	1330	2730	905	1890	630
Average	2570	0.746	513.0	6350	2480	5000	1960	3670	1445	2790	1130	4580	1790	3620	1420	2710	1060	2070	850

TABLE 8—CONTINUED

Laboratory No.	Strength	n	k in 1000 lb. per sq. in.	Secant Modulus in 100-lb. per sq. in. for Different Percentages of Ultimate Strength, and E/Strength								Tangent Modulus in 1000 lb. per sq. in. for Different Percentage of Ultimate Strength, and E/Strength							
				E 5%	E/S 5%	E 10%	E/S 10%	E 25%	E/S 25%	E 50%	E/S 50%	Et 5%	E/S 5%	Et 10%	E/S 10%	Et 25%	E/S 25%	Et 50%	E/S 50%
AGGREGATE 0-¾-IN.; AVERAGE PASTE CONTENT 0.38																			
2280	2060	0.769	328.0	3680	1790	3005	1460	2320	1130	1850	900	2830	1380	2300	1120	1780	870	1420	690
2503	3010	0.801	634.0	3880	1290	3750	1250	2230	1075	2900	965	3190	1060	3010	1000	2590	865	2320	770
2537	2460	0.758	398.0	5270	2150	4230	1725	3150	1280	2560	1040	4000	1630	3200	1300	2380	970	1940	790
2447	3200	0.688	214.0	5730	1800	4150	1300	2760	865	2000	630	3940	1230	2860	895	1900	595	1380	432
3452	3060	0.791	597.0	5270	1720	4400	1440	3430	1120	2860	935	4170	1360	3480	1140	2710	885	2260	740
3538	2680	0.791	496.0	4320	1610	3620	1350	2820	1050	2370	885	3420	1270	2860	1070	2230	830	1870	700
2774	3000	0.836	875.0	4850	1620	4170	1320	3480	1170	3030	1010	4060	1360	3480	1170	2910	975	2530	845
3559*	2920	0.694	263.0	7180	2460	5220	1790	3460	1180	2560	880	4980	1710	3640	1210	2400	820	1780	610
3633*	3370	0.688	273.0	7770	2310	5720	1700	3760	1115	2790	830	5350	1590	3940	1170	2590	770	1920	575
Average	2850	0.755	453.0	5330	1860	4250	1490	3160	1110	2540	895	3970	1395	3180	1190	2380	840	1930	680
AGGREGATE 0-⅜-IN.; AVERAGE PASTE CONTENT 0.45																			
2281	2220	0.764	282.0	3245	1460	2610	1175	2090	940	1580	710	2480	1115	1990	895	1600	720	1210	545
2605	2860	0.796	501.0	4030	1410	3360	1175	2660	930	2240	785	3210	1120	2680	935	2120	740	1780	620
2538	2410	0.778	440.0	4540	1880	3710	1540	2850	1180	2310	960	3540	1460	2890	1200	2220	920	1800	750
2448	3410	0.834	695.0	3550	1040	3130	920	2540	745	2240	660	2960	870	2610	765	2120	620	1870	550
3453	3250	0.606	121.0	9120	2800	5800	1790	3220	990	2030	625	5530	1710	3520	1080	1950	600	1230	380
3539	2660	0.722	263.0	4740	1260	3690	980	2620	700	2010	540	3420	915	2660	710	1890	505	1450	387
2775	3370	0.779	522.0	5180	1540	4220	1250	3240	960	2670	795	4030	1190	3210	950	2520	750	2080	620
3560	3050	0.691	232.0	6100	2000	4420	1450	2930	1280	2160	710	4220	1380	3050	1000	2020	660	1490	490
3634*	3620	0.606	130.0	9320	2550	5880	1630	3230	895	3050	845	5580	1550	3560	985	1950	540	1850	510
Average	2980	0.730	354.0	5530	1770	4100	1325	2820	960	2240	730	3870	1250	2910	965	2040	670	1630	537

* Doubtful data.

INGE LYSE* (*By Letter*).—Mr. Powers is to be commended for attacking a very difficult and important problem. The tendency to use large size aggregate in concrete structures has raised the question of developing a suitable means of sampling and testing this concrete. On most jobs the size of specimens is limited to 6 x 12-in. or 8 x 16-in. cylinders. Experience and tests have led to the conclusion that the maximum size of gravel aggregate should not exceed ¼ of the diameter of the cylinder. This requirement makes it necessary to use other means in testing concrete containing aggregates too large for the ordinary sizes of cylinder. Mr. Lyse.

The problem which Mr. Powers has attempted to solve is the development of a method by which a representative sample of concrete made with 9½-in. aggregate can be tested in the standard 6 x 12-in. cylinder. The most obvious method is the one he has followed of removing the larger particles of coarse aggregate. This procedure, however, changes the consistency of the mix and if carried too far may result in erroneous conclusions as to the actual strength of the concrete containing all the coarse aggregate.

With ordinary concrete mixes the removal of aggregate above 1½ inch in size will not change the consistency enough to affect the strength appreciably. Before an appreciable change in strength will occur, the consistency must be changed to such an extent that segregation of water from the paste takes place. When segregation occurs the water accumulates at the top of the specimen and the water-cement ratio of the paste is therefore changed. It is the water retained in the paste at the time of hardening which determines the strength and not the amount of water used when the concrete is mixed. Mr. Powers very properly mentions this fact in his discussion of the data, but since the amount of segregated water was not observed it is impossible to judge exactly the extent to which the loss of water affected the strengths he has reported. The observed slumps, however, give some information as to the extent of the segregation in his tests. His data show that the slump changed from 4 in. to 10 in. when all aggregate above ⅜ in. was screened from concrete made with aggregate of 3 in. maximum size. Such wide changes in the consistency as indicated by these slumps will materially affect the quality of the paste.

Ordinary concrete from well-graded aggregate does not segregate for consistencies corresponding to slumps of 6 in. or less. Mortar, however,

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starts to segregate at a consistency corresponding to a slump of 3 to 4 in. The exact consistency at which segregation takes place is determined principally by the grading of the sand. The coarser the sand, the earlier the segregation takes place and the greater its amount. In tests under way at the Research Laboratory of the Portland Cement Association, it has been observed that if a large change in consistency is not corrected, for in some way there will be a very considerable difference in the strength of the specimens due to the change in the net water-cement ratio of the paste.

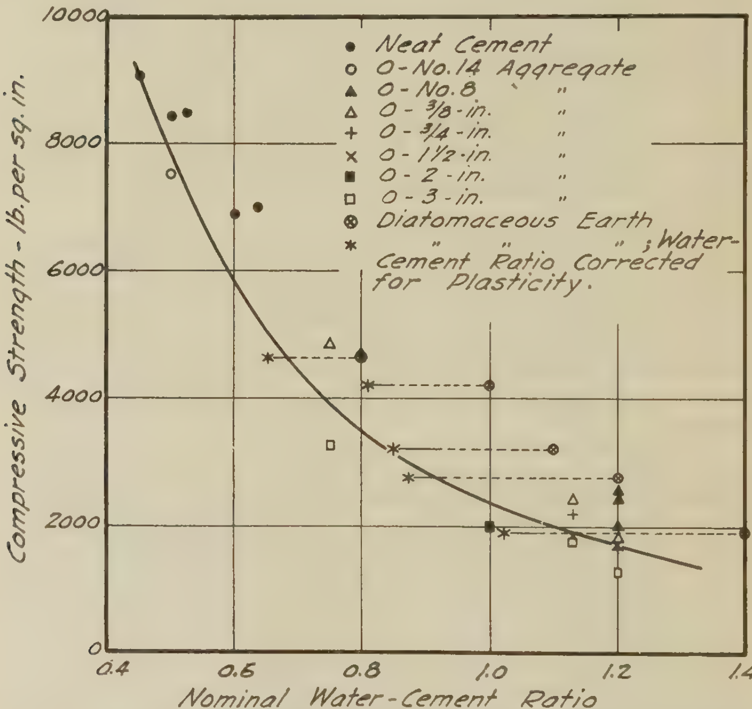

FIG. 1.—RELATION OF WATER-CEMENT RATIO TO STRENGTH OF CONCRETE, MORTAR AND CEMENT PASTE OF VARYING CONSISTENCY.
Data from Table 1 of T. C. Powers' paper on "Concrete Studies of Bull Run Dam."

The data in Table I of Mr. Powers' paper have been plotted in Fig. 1. The scattering of the points is probably due largely to the fact that most of the tests are based on only one specimen. It is believed to be due in part to the segregation of mixing water from the wet mixtures, and to the use of too large aggregate in some of the 6 x 12-in. specimens. For the mixes containing diatomaceous earth the points are separated from the curve by a distance corresponding approximately to the amount of water required to maintain a constant plasticity which Mr. Powers states

corresponded to about twice the absolute volume of the admixture. Since the diatomaceous earth has a specific gravity of about 2.0 this addition of water would be equal to the weight of the admixture.

The curve in Fig. 1 can be taken as a satisfactory representation of the water-cement ratio strength curve for the materials used in these tests. It will be noted that the strengths for the mixes containing aggregates of 2 and 3 in. maximum size are below the curve, while most of the neat cement and mortar specimens had strengths that lie above. The mixes containing diatomaceous earth agree fairly well with the curve when the water-cement ratio is corrected for the water required to maintain constant plasticity.

The difference between the strength of the 6 x 12-in. concrete cylinders made with 2 and 3-in. aggregate in Mr. Powers' tests and the strength indicated by the curve in Fig. 1 corresponds approximately to the reduction in strength which we have found with these sizes of aggregate in this size of cylinder. Also the excess strength of the mortar and neat cement specimens in Fig. 1 can be accounted for to a large extent by the loss of water due to segregation. Leaky molds will also decrease the water content, and where an accurate relation between water-cement ratio and strength is desired, sealed molds should be used and measurement made of any segregated water.

Fig. 2 shows some of the results of a series of tests made in the Research Laboratory of the Portland Cement Association for the purpose of studying the effect of segregation of mixing water on the strength of concrete and mortar. The fine aggregate consisted of Elgin sand graded 0 to No. 4 and the coarse aggregate of Chicago limestone graded up to 3 in. The size of the cylinders was varied from 12 x 24-in. to 2 x 4-in. in order to obtain a comparison between the strength of the concrete when tested in a cylinder of suitable size without removing any of the large particles of aggregate and that of the same concrete when molded in smaller specimens after removal of all or part of the coarse aggregate particles. In molding the 6 x 12-in. cylinders all aggregate above the 1½-in. size was removed, while all aggregate above the No. 4 sieve was removed in molding the 4 x 8-in., 3 x 6-in., and 2 x 4-in. cylinders. All molds were sealed so that no leakage occurred and the amount of water which accumulated on the top of the specimens was measured.

The concrete mixes showed practically no segregation and there was very little difference in strength due to differences in size of cylinder. The removal of aggregate between the 1½ and 3-in. sizes did not seem to affect the strength. However, when all the aggregate above the No. 4 sieve was removed a great change in consistency resulted, considerable segregation of water took place, and the mortar specimens showed considerably higher strength than the concrete specimens, as may be seen in Fig. 2 for both the 7 and 14-day tests. The plotted values are the average of the four specimens for each size of cylinder. It will be noted that the strengths from the specimens with all the coarse aggregate screened out, plotted in terms of the original water-cement ratio, are considerably

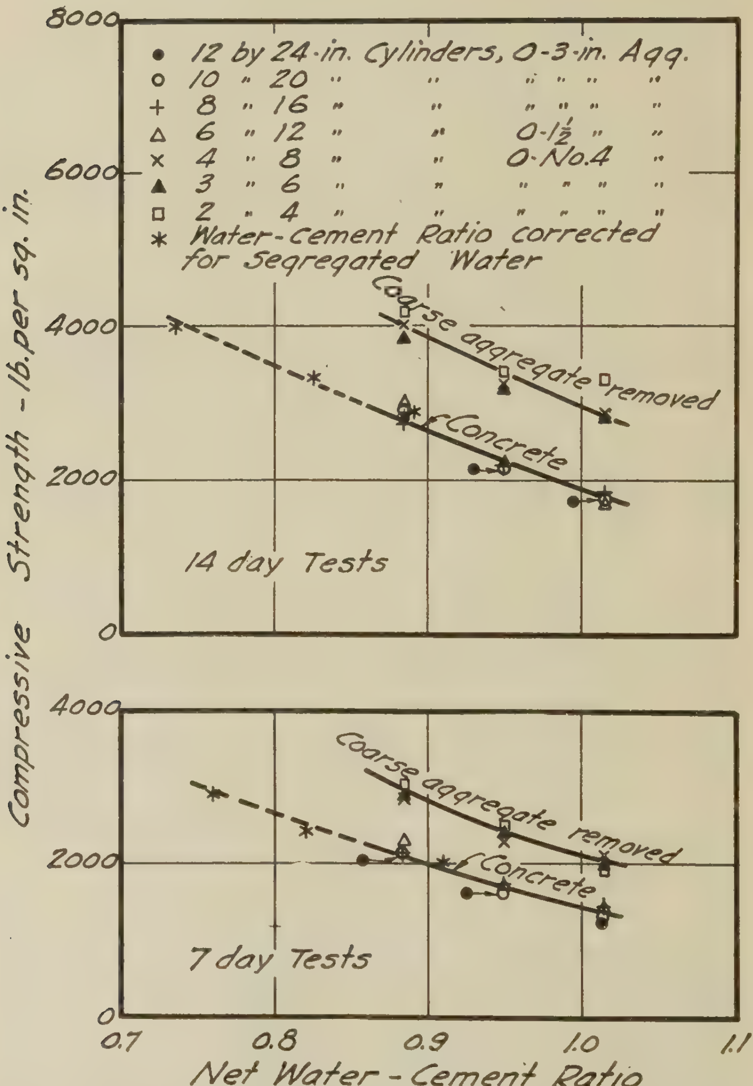

FIG. 2.—EFFECT OF SEGREGATION OF MIXING WATER ON WATER-CEMENT RATIO AND STRENGTH OF CONCRETE AND MORTAR.
Data from Series J-110, Group II; Research Laboratory, Portland Cement Association.

above those for the tests of mixes containing the coarse aggregate. At 7 days the strength of the mortar was about 700 lb. per sq. in. greater than the concrete and at 14 days the difference had increased to about 1100 lb. per sq. in.

The dotted portion of the lower curve of Fig. 2 was obtained by extending the curve through the points representing the tests of the specimens with the coarse aggregate screened out when plotted against the water-cement ratios corrected for the measured amount of segregated water. Thus it is seen that the water-cement ratio is an accurate measure of the strength of both the concrete and mortar, provided the true water content in the mass at the time of hardening is used.

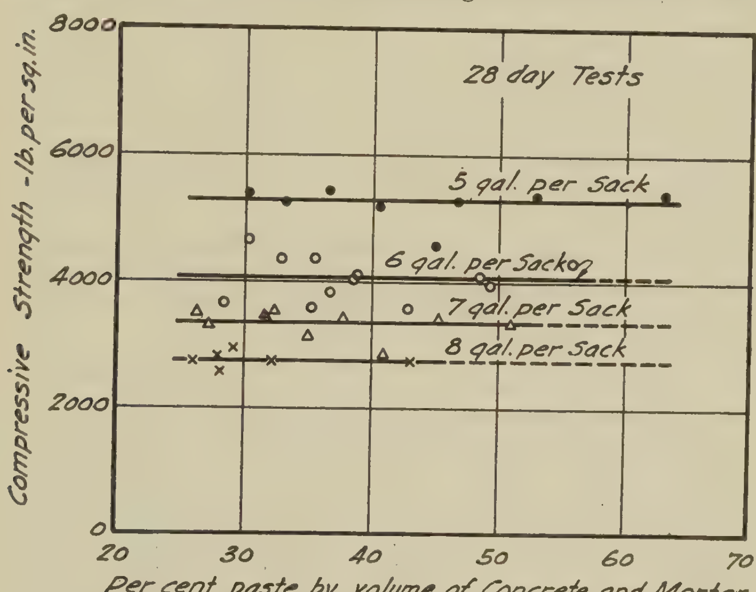

FIG. 3.—EFFECT OF AMOUNT OF PASTE ON STRENGTH OF CONCRETE AND MORTAR OF GIVEN NOMINAL WATER-CEMENT RATIO.
Data from Series 186, Research Laboratory, Portland Cement Association.

Further evidence that the amount of paste in the mixture does not alter the strength so long as the quality of the paste remains unchanged will be found in Fig. 3, which is based on tests of concrete mixtures of constant nominal water-cement ratio, but having a variable proportion of fine to coarse aggregate. Each point in this figure represents the average of 5 specimens made from workable mortar or concrete. The results have not been corrected for water absorbed by the aggregates and some of the irregularities would be eliminated by such a correction. The results indicate that the amount of paste has but very slight effect on the strength of concrete and mortar. Specimens with 30 per cent

paste show about the same strength as specimens with 63 per cent paste so long as the water-cement ratio is maintained constant. A large difference in strength, however, is noted both for concrete and mortars of different water-cement ratios.

The well-known German concrete investigator, Prof. Otto Graf, Stuttgart, has come to the same conclusion. His method of determining the strength of concrete by the strength of the mortar in the concrete is simply another application of the water-cement ratio law because he makes no change in the paste of the mortar by additions of coarse aggregate. In a paper read before the German Concrete Institute convention, March 27, 1928, Prof. Graf presented the data shown in the accompanying table.

Tamped Concrete, Crushed Limestone Aggregate			Plastic Concrete, Rhine Gravel Aggregate		
Mix by Volume	Compressive Strength at 28 days, lb. per sq. in.	Sacks Cement per cu. yd. Concrete	Mix by Volume	Compressive Strength at 28 days, lb. per sq. in.	Sacks Cement per cu. yd. Concrete
1:2:0......	5120	13.1	1:2:0......	4420	11.7
1:2:1......	5400	10.2	1:2:2......	4450	7.0
1:2:2......	5670	8.4	1:2:3½...	4180	5.6
1:2:4......	5760	6.1	1:2:5......	3840	4.5

It is seen that the strength of tamped concrete increases slightly as a larger amount of coarse aggregate is added to the given mortar. This increase in strength is probably due to the water absorbed by the added aggregate. The plastic concrete showed a fairly constant strength until the mix became harsh due to under-sanding. Prof. Graf's table shows clearly that the amount of paste has very little effect on the strength of concrete. The amount of paste, which is proportional with the number of sacks per cu. yd. concrete as long as the water-cement ratio is kept constant, is decreased more than 50 per cent without reducing the strength of the specimens.

The data presented in this discussion indicate that the strength of the concrete is fixed by the actual water-cement ratio of the cement paste. The amount of paste has little or no effect on the strength of the concrete or mortar so long as the actual water-cement ratio of the paste has been maintained, and the mix is plastic and workable. The consistency and workability, however, are affected largely by the amount of paste and also the extent of segregation of water which produces changes in the actual water-cement ratio and thereby affects the strength of the concrete. In order to obtain a true water-cement ratio strength relation it is, therefore, necessary to deal with plastic and workable mixes having little or no segregation, or measure the amount of segregated water and make allowance for this in interpreting the results.

Joseph A. Kitts* (*By Letter*)—Compression tests are so erratic that conclusions based on a few test specimens are likewise apt to be contrary to the fact. Very minute defects in the caps of cylinders cause very large errors in the compression test results. This is thoroughly proved in Bulletin 14, Structural Materials Research Laboratory, Lewis Institute, published under date of April, 1925. **Mr. Kitts.**

We have made many hundreds of comparative tests of concretes with large maximum size aggregate and have found: (1) that cylinders with diameters in excess of 3 times the maximum diameter of particles give greater unit strengths for the same concrete than cylinders with diameters less than 3 times the maximum diameter of particles and of the same relative length to diameter; (2) that small cylinders (less than 3 times the maximum diameter of aggregate) give better results when particles over ⅓ the diameter are thrown out; and, (3) that large cylinders give higher unit strengths than small cylinders where the maximum particles, over ⅓ the diameter of the cylinder, in each case, are thrown out. The few tests of the author appear to show the contrary result.

This contrary result may be due to greater time of mixing, evaporation of water and more complete hydration of the cement during the screening and remixing process, as well as to error due to the small number of specimens. Of course the important consideration in the use of large maximum size of aggregate in mass concrete is that it gives greater strength per dollar cost of materials. This fact is well established.

Mr. Powers' Fig. 1 shows a strength of 1800 lb. per sq. in. for $W/C = 0.5$ and $p = 0.05$. As the absolute volume of cement is equal to the absolute volume of water in this case, a cement content of 2.5 per cent with a water content of 2.5 per cent and an aggregate content of 95 per cent certainly cannot show a strength of 1800 lb. per sq. in. I am quite certain that the W/C curves start tangent to the abscissa rather than tangent to the ordinate as shown, the strength then rising at an increasing rate until the paste approaches the volume of voids in the aggregate, after which it rises at a decreasing rate as the paste increases.

The endeavor to find an equation of strength more absolute than the Abrams water-cement formula is commendable. There is no doubt but that many factors affect the strength other than the water-cement ratio. One must expect to do a very large amount of testing, research, analysis and differentiation, however, before any definite conclusions may be made and the absolute formula of strength may not be expected to be a simple one involving only one or two variables. There are at least three variables—aggregates, cement and water—and there are perhaps six or more.

That being the case, at the present time the only solution of the situation is to make comprehensive tests in absolute basis mixtures of the job aggregates and cement, varying the grading and cement content for the particular workability required, graphically and mathematically analyzing the results, selecting the basic mixture with the optimum of

* Consulting Concrete Technologist, San Francisco, Calif.

required qualities and maintaining that basic mixture as the aggregates and cement vary.

Mr. Powers gives the equation used for grading as $p = (d/6)^{0.45}$, and it is not clear whether p is the proportion by dry-rodded, loose-moist, inundated or absolute volume or by weight.

Mr. Powers declares that there is no relation of strength to density. It should be quite evident, however, that, as increments of paste are added to a mixed aggregate, both the density and strength of the concrete increase as the paste tends to fill the voids in the aggregate. When at first the paste content and density are both increasing, the strength increases rapidly; when the paste content is increased further while the density is decreasing, the strength increase is very much reduced and the rate of strength increase decreases with the decrease of density. This is abundantly proven in Abrams' Bulletin 4 and by the work of Talbot & Richart in their Bulletin No. 137. High strength depends either upon high density or high cement content. High strength by high density is economical, while high strength by high cement content is wasteful.

As Mr. Powers states, there does not appear to be any line of demarkation between sand-cement mortar and concrete. There is a very consistent relation, however, between strength and sand-cement ratio, for a given cement, aggregate and workability of mix, in the straight line form $K = A - BS/C$ in which K is the tensile or compressive strength of the mortar or concrete, A is the cement constant, B is the sand constant and S and C are the absolute volumes of sand and cement, respectively. Like the water-cement-strength equation, this is closely approximate within certain reasonable limitations and for the given conditions of aggregate, cement and workability.

While the writer cannot agree with many of the author's conclusions, he does wish, however, to encourage individual research of this original character. The author has done a lot of hard work along a new line of thought and the record is a valuable one.

Mr. Derleth. C. P. Derleth (*By Letter*)—Mr. Powers is to be congratulated on the completeness and comprehensive presentation of the data included in his paper. Tests as carried on at Washington University, St. Louis, tend to confirm that the water-cement-strength relationship is contingent upon maintaining a uniform relationship between water-ratio, paste content and upon the degree of homogeneity, or plasticity.

Exception is taken to Mr. Powers' statement that "the gain in strength and water-tightness due to the use of diatomaceous earth is *not due to absorption.*" A direct test using saturated lime water, the vehicle of concrete, will show that the absorption is more than three times the weight of admixture. Water absorption and water retention capacity are practically identical. The increase in yield under fixed aggregate conditions is entirely due to the increase in volume of paste content had from the admixture plus that additional lime water which the admixture absorbs and retains in the mass until the initial set has taken place. The

per cent increased yield of paste or the resulting smaller per cent increase in yield of concrete can be computed for the recommended quantities or weights of various admixture materials. The colloidal silicas have great capacity for the retention of water.

The use of a highly absorbent admixture in the concrete due to its water retention capacity has an effect of reducing actual density though, as brought out in Mr. Powers' paper, gives a more impermeable concrete. The penetration of water into concrete capillaries is governed by the law of capillarity. Although the per cent porosity is increased, if void sizes are smaller and void distribution more uniform, a more water-tight concrete will result. It would seem to the writer that statements three and four are contradictory since the use of an absorbent admixture requires additional mixing water in the dryer mixes only to compensate for the drying effect of the admixture. This does not hold true in the wetter (segregating mixes) since there is normally sufficient or more than sufficient water to satisfy the absorption requirements of the admixture used, converting this normally fluid water into an increase in paste content.

The effects mentioned of segregation on area of bond to aggregates and steel have been reported on by Gilkey and others as being the result of sedimentation of the heavier component of the mortar (cement) induced due to the settling of the heavier aggregates, this causing upward displacement of the lighter constituent, the water.

The effect of colloidal silica and other highly absorbent admixtures seemingly to increase the viscosity and suspending characteristics of the mortar has been determined an effective means of eliminating the inevitable gravity segregation of materials of widely different specific gravities comprising the concrete.

Commenting on the effect of additional mixing water on strength it has been shown by Conahey that while the water absorbed by a limited amount of admixture does increase the actual water-cement ratio of the set mix, the effective or available water-cement ratio remains unchanged.

T. C. POWERS (*By Letter*)—The discussion of the writer's paper has been of extreme interest, but at the same time somewhat confusing. Mr. Derleth states that tests at Washington University tend to confirm our work. Mr. Kitts finds the opposite, and Mr. Lyse believes that our results were influenced by variation in the water-cement ratio, and by the ratio of the size of the largest particle to the diameter of the test specimen. Mr. Powers.

Since the presentation of the paper, the writer has done much more work along the same line, but having as an objective the study of the effect of paste content on the modulus of elasticity. In many of the later tests, very careful determinations of the *net* water ratio were made. Time and space will not permit details, but, in general, we found that the differences in strength between the 0–1½-in. and 0–3-in. cylinders were *greater* than first reported, and that the differences between the smaller sizes were *less* than first reported.

The term "workability" is not definitely enough defined to permit its

use in classifying concrete mixtures except in a very general way, but many tests beside our own have shown that the strength of mixtures in which there is sufficient paste to envelop the aggregate is influenced by the quantity of paste as well as by the quality.

A study of the stress-strain curves from our elasticity tests suggests the reason for the difference in strength developed by the mortar specimen, for example, and the 3-in. specimen sampled from the same mixture. We now have about 200 tests which show *without exception* that, even after correcting for unequal losses of water through leakage and segregation, the 0–3-in. specimens test from 50 to 60 per cent of the strength of the mortar specimens. The fact that these tests were all made on 6 x 12-in. cylinders may be of significance.

Concrete may be thought of as being made up of two components, paste and aggregate. Aggregate particles, large and small, do not touch each other, being held in suspension by the paste. When such material is stressed, the resulting deformation should be the resultant of the deformations of the two components, paste and aggregate. Paste is known to have a very low modulus of elasticity, depending on the w/c and effective age; our basaltic aggregate has a very high modulus of elasticity, about ten million. When a mortar specimen and a 0–3-in. specimen are stressed, the resulting deformations are proportional to their respective paste contents at *very low stresses only.* As the stress increases the 0–3-in. (low paste content) cylinder begins to deform at a relatively higher rate, reaching its yield point at a relatively low stress and failing at about one-half the ultimate strength of the mortar cylinder.

This suggests the theory that when the stiffer component is present in relatively large masses, the concrete does not act as a homogeneous material. That is, an elemental column in one part of the cylinder may be 50 per cent while another parallel column may be but 25 per cent due to the necessarily unequal distribution of the large aggregate particles. When loaded, the low paste content column will deform less than the higher paste content column, and will therefore receive a disproportionate share of the load, thus increasing the stress in the paste of that column. Under such conditions one would expect the cylinder to fail at a lower stress than one made of pure paste of the same quality, or one having a much higher paste content such as the mortar cylinder.

This theory suggests that the ratio of the size of the particle to the size of the cylinder is an important factor.

Mr. Kitts makes a point of the few number of tests made, and of the erratic nature of compression tests. In testing over 3000 cylinders at the Bull Run Dam, cylinders from the same batch varied about 3 per cent from the average. We therefore expect most of our cylinders to be within 3 per cent of the true value. Routinized procedure and carefully developed technique does much for the much maligned compression test.

Mr. Kitts also takes exception to Fig. 1, pointing out that 5 per cent paste, 95 per cent aggregate, $w/c = 0.50$ is shown as being 1800-lb. concrete. The writer acknowledges fault on two counts: first, for not stating

that the curves are only for concretes in which the paste is present in sufficient quantity to envelop the aggregate, and, second, for extending the curves beyond the realm of experiment data. If such a mixture as the above could be made, I am not at all certain what the strength would be.

Regarding the relation of strength to density, I would refer Mr. Kitts to data recently published by the Portland Cement Association. It is also well to mention that any mixture in which there is not enough paste to "fill the voids" should not be considered as concrete.

Mr. Derleth is using the term absorption in a different sense than did the writer. As he uses the term, all the water held on the surfaces of a material by adhesion and surface tension is absorbed water and not effective in the mixture. The term absorption as commonly used with reference to aggregates refers to the water taken into the stone by capillary action. Once inside the particle it is definitely out of the mix because it is held in a case fully as strong as the paste itself. But even if the water were drawn into a cellular diatom to the exclusion of the cement, it might as well be free water in the paste; for such a thin, frail, shell filled with water could offer little resistance to stresses. The writer has produced precisely the same results as obtained by the use of diatomaceous earth by using appropriate amounts of such materials as rock flour (basalt), volcanic dust (not puzzolanic), or clay, all of which are definitely not cellular.

If an admixture effects a mixture by virtue of absorption, then to add diatomaceous earth to a rich mix without adding water should increase the strength. We know that this is not the case. Only the leaner, semi-plastic mixtures will show an increase in strength, and the leaner the mixture, the more effective the admixture.

The constructive criticism which the paper has brought out is very gratifying, and the writer feels well repaid for his efforts.

Researches on Concrete Materials and on Plain and Reinforced Concrete

Submitted by Committee E-3 on Research

Introduction—This report, the fourth to be compiled since the reorganization of this committee in 1925, has been prepared in much the same way as the previous ones, with the exception, however, that only new projects or projects which were completed during the year have been reported. This procedure has eliminated a large amount of duplication by omitting summaries of investigations which have been under way for several years, and which, being in the nature of long-time tests, will not be completed for several more. The response to the Committee's questionnaire has been very gratifying, and since the replies were more informative than in previous years, the preparation of brief summaries on current researches was greatly facilitated.

For convenience of reference, the subject-matter of this report is presented in the following sections:

I. *Researches on Cement*—These cover such subjects as the following: workability of cements and development of methods of comparison; development of methods of testing cement to replace those of the present A.S.T.M. Specifications; study of the physical characteristics of hot cement; strength of cement grout as used in cementing oil wells; constitution of portland cement; properties of special cements.

II. *Researches on Aggregates* including studies on durability of aggregates; relation between grading and void content of aggregate; relation between grading and strength and yield of concrete; effect of deleterious materials in aggregates; general investigations of the role of aggregates in the finished concrete.

III. *Researches on Plain Concrete*—Most of the investigations reported are on plain concrete and include studies of the effect of admixtures and deleterious substances on the strength and other properties of concrete; autogenous healing; methods of curing; tests of arch dams; durability; elastic properties; extensibility and fatigue; fire resistance; flow under load; freezing and thawing; pavement construction; permeability; proportioning; methods of testing; compressive and flexural strength; temperature effects; time of mixing; volume change; waterproofing; wear; investigation of structures in service.

IV. *Researches on Reinforced Concrete*—This section deals with such subjects as behavior of reinforced concrete arches under load; tests of

reinforced concrete beams to determine the effect of variation in spacing of vertical stirrups on strength; the efficiency of hooks and anchorage of reinforcing bars; effect of various percentages of steel; tests of beams under continued load; studies of bond; and tests of bridges.

V. *Suggested Researches on Concrete and Related Subjects*—Under this heading is grouped a list of subjects which have been submitted as timely topics on which research is needed. These include such subjects as effect of dirty aggregates on strength and durability of concrete; effect on strength of arrangement of aggregate particles in concrete; development of a satisfactory workability test for concrete; effect of admixtures on watertightness of concrete; study of vibrations of reinforced concrete members; and the use of welded connections between reinforcing bars as a possible means of increasing strength.

VI. *References to Papers and Reports on Researches Published during 1928*—This section of the report gives an extensive list of references to important articles on concrete and related subjects published during the year in the United States and in foreign countries. It is believed that these references will be of considerable assistance to the members of the Institute as they comprise a fairly complete bibliography on the literature published during 1928. The wide variety of subjects listed shows the great amount of research being carried out in the field of concrete and reflects the continued interest in this material.

The Committee again expresses its appreciation to the many organizations and individuals who have cooperated by furnishing information on their various researches. Without their aid it would be impossible to compile these reports.

This report has been submitted to letter ballot of the Committee, which consists of 12 members, of whom 11 have voted affirmatively, none negatively, and 1 has refrained from voting.

H. F. GONNERMAN, *Chairman*
F. E. RICHART, *Secretary*

Researches on Concrete Materials and on Plain and Reinforced Concrete

In general, reports giving data of the researches listed in this report are not available. In case a report of an investigation has been published, giving the methods of test and the results wholly or in part, reference is made thereto. The conclusions given for some of the investigations are those of the investigator, and do not necessarily represent the views of the Committee.

I. CEMENT

Workability of Portland Cement (National Bureau of Standards, Washington)—The purpose of this investigation was to determine the workability of different cements and to develop means and methods of comparison. Certain devices and methods have demonstrated differences between various cements, and some of the devices have proved capable of reasonably good repetition of results. The mortar tests to date have covered only a narrow field due to limitation in size and power of the apparatus. The indications appear reasonable, and the tests will be carried farther with a larger apparatus of modified design.

Testing of Portland Cement (Engineering Dept., City of Los Angeles)—A pressure stroke method for the compaction of 2 x 4-in. cylinders was developed to be substituted for tamping in the testing of cement under A.S.T.M. Tentative Specification C9-16T. The method investigated produced specimens of very consistent strength, largely eliminating personal equation.

Correlation Study of Strength Tests for Portland Cement (Maine Technology Exp. Station, Orono)—The purpose of this study was to determine the relation between the reliability of standard briquets, 2-in. cubes, and 2 x 4-in. cylinders, made from 1:3 Ottawa sand mortar. Results of this investigation published in Bulletin 22 of the Maine Technology Experiment Station.

Study of Hot Cement (Tennessee Highway Department, Nashville)—An investigation of the characteristics of hot cement when ground from (1) raw clinker direct from kiln, (2) 50 per cent raw, hot clinker and 50 per cent aged, cooled clinker from stockpile, and (3) clinker aged and cooled in stockpile. 2 x 4-in. cylinders, standard briquets, and pats were made from cement ground from each of the clinkers at temperatures varying from 375 deg. F. to normal by 25 deg. decrements as the cement cooled immediately after manufacture. Standard strength tests were made on briquets and cylinders at 7 and 28 days, and time of setting and soundness tests were made on pats. The results showed that so long as the cement meets the physical requirements of the A.S.T.M. Specifications, the temperature at which the cement is received on the construction job is not important. There was no material difference in either compressive or tensile strength, or in other characteristics. A slightly earlier setting time was found, but this is regulated under the A.S.T.M. Specifications.

Plastic Mortar Compression Tests for Cement (Lehigh Portland Cement Co., Allentown, Pa.)—The inadequacy of the standard 1:3 mortar tension test led to intensive search for a test that would better indicate the concrete-making value of a cement. The most promising solution of the problem thus far obtained is a compression test on 2-in. cubes made from plastic mortar of cement and sand having a definite water-cement ratio and a definite consistency. In studies extending over a year, a definite gradation and amount of sand (in relation to the quantity of cement) were used with gratifying results.

Cooperative studies with other laboratories under the auspices of the A.S.T.M. are now being carried out to determine the limitations of the new test in respect to concordance of results obtained by different operators in different laboratories, and in respect to the necessity for uniformity in type, proportion, and gradation of sand. The proposed test and preliminary results obtained using it were presented in a paper by E. M. Brickett on "A Plastic Mortar Compression Test for Cement." (See *Proc.* A.S.T.M., v. 28, 1928.)

Strength Developed by Cement Grout as Used in Cementing Oil Wells (Lehigh Portland Cement Co., Allentown, Pa.)—The demand for definite data on the strength of cement grout under conditions existing at the bottom of oil wells led to an extensive series of tests in which the peculiar conditions of temperature and pressure encountered in the field were simulated in the laboratory. An apparatus was constructed in which the test cylinders could be subjected to pressures up to 3000 lb. per sq. in., and temperatures up to 175 deg. F. immediately after placing, and maintained under these conditions as long as desired. It was found that a very high acceleration in the rate of hardening was obtained under a pressure of 2100 lb. per sq. in. and a temperature of 135 deg. F. (at which most of the tests were made). The water-cement ratio, however, was the most important single factor affecting the strength developed, and the control of the consistency at the lowest permissible water content was indicated as being fundamentally the proper basis of procedure for certainty in results and saving of time. One point of interest in connection with these tests was that a water from the oil fields, heavily laden with sodium and calcium chlorides in solution, gave higher grout strengths than local drinking water, with or without calcium chloride.

Physical Properties of High Alumina Cement Concrete (University of California, Berkeley)—These tests are being made on 6 x 12-in. concrete cylinders to determine strength and elastic properties, and on 3 x 3 x 40-in. bars to investigate volume changes due to variations in moisture conditions and expansion due to variations in temperature.

Investigations of Lumnite Cement Concrete (Washington University, St. Louis, Mo.)—These tests were made in two series, the first of which included 175 standard compression cylinders of 1:2:4 concrete with varying water-cement ratios, cured after the first 24 hr. in open air subject to varying outdoor temperatures and humidity. Tests were made at 24 hr., 7 and 28 days, 3. 6, and 9 mo., and 1 and 2 yr. Maximum strength in all cases was reached at the 28-day period, after which there was a marked decrease in strength until the 6-mo. period; from then on to 2 yr. there was no marked change. In all cases, strength at 6 mo. was less than the 24 hr. strength.

The second series included 200 standard compression cylinders of varying proportions and water-cement ratios, cured both outdoors and in the testing room. Tests were made at 24 hr., 7 and 28 days, 3, 6, and 9 mo., and the 1-yr. tests are due early in 1929. Maximum strengths were reached at varying times from 7 days to 3 mo. In all cases, after the maximum strength was reached, there was a gradual decrease, the 9-mo. strength being less than that at 7 days.

Investigations on Lumnite Cement (Atlas Lumnite Cement Co. Fellowship, Bureau of Standards, Washington, D. C.)—Appreciating that high alumina cements differ from portland cements both in chemical composition and in physical properties, and that a correct knowledge of these differences would be of value both to the producer and the consumer, the Atlas Lumnite Cement Co. established a Fellowship at the Bureau of Standards for the sole purpose of investigating the properties of high alumina cements.

The work of this fellowship was begun in the spring of 1925. The principal subjects under investigation include the following:

(1) Effect of different water-cement ratios and different conditions of storage on the compressive strength and durability of alumina cement concrete;

(2) Temperature evolved during hardening and its relation to the compressive strength at different ages under different conditions of mixing and curing;
(3) Volume changes of high alumina cement during hydration and after hardening;
(4) Effect of initial temperatures on the hardening and durability of the concrete.

The results of these investigations will be published from time to time when sufficient information and data are obtained to warrant drawing conclusions.

Effect of Moist Closet Curing on Strength of Standard Cement Mortar Briquets (Indiana State Highway Commission, Indianapolis)—The results of this investigation indicate that as far as practical routine testing is concerned the effect of reasonable temperature variations in moist closets on strength of standard briquets is slight at an age of 7 days, and has entirely disappeared at age of 26 days. If special early age testing is to be done, this factor becomes rather important and should be carefully controlled.

Cooperative Tests of 8 Cements by 8 Laboratories (Sub-Committee VII of A.S.T.M. Committee C-1 on Cement)—The purpose of these tests is to investigate the possibilities of plastic mortar compression test on a 2-in. cube as a means of evaluating the strength of a cement in concrete. Eight laboratories are cooperating in the tests which include compression tests of 6 x 12-in. concrete cylinders, compression tests of 2-in. mortar cubes made from the fine aggregate used in the concrete, compression tests of 2-in. cubes using a run-of-mine Ottawa silica sand and tension tests of 1:3 standard sand mortar. All compression specimens will be of the same nominal water-cement ratio and tests will be made at 2, 4, 7, and 28 days. It is planned to report the data at the Annual Meeting of the A.S.T.M.

Constitution of Cement (Portland Cement Association Fellowship, Bureau of Standards, Washington)—A continuing investigation of physico-chemical phenomena associated with cement. The work of the past year has included studies of phase equilibria in a number of systems, chemical reactions of setting and hardening, effects of salt solutions on the constituents of set cement, influence of compound composition on cement value and influence of fineness and temperature of burning on cement value. The phase equilibria studies have reached the point of being applied to the burning of carefully controlled mixtures in a laboratory rotary kiln, as a first step toward plant scale application.

II. AGGREGATES

Study of the Durability of Concrete Aggregates (National Bureau of Standards, Washington)—The purpose of this investigation was to determine the extent that lack of durability of the aggregate is responsible for the breaking down of concrete due to weathering. Thirty-six varieties of the most common representative coarse aggregates throughout the United States were collected and tested. Samples of each type of aggregate were subjected to the sodium sulfate, sodium chloride, boiling and drying, and freezing and thawing tests. After 150 cycles of these tests the aggregate was made into nominal 1:2:4 concrete, formed into 6-in. cubes, and tested at the age of 3 mo.

Investigation of Slag and Gravel Aggregates (University of Tennessee, Knoxville)—This study was undertaken to obtain a relationship between slag and gravel concrete strengths. Only a small amount of work has been done to date, and no results are yet available.

Relation between Grading of Coarse Aggregate and Strength of Concrete (National Bureau of Standards, Washington)—This investigation was a check on the sufficiency of the table of mixes in the 1924 Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, as well as a study of the effect of systematic variation of grading of coarse aggre-

gate. Specimens were made with 72 mixes taken from a table in the Joint Committee Report, and gravel, crushed slag, and crushed limestone were used with 4 different cements in widely varying proportions. The resulting strengths conformed to the water-cement ratio law when the quantity of sand exceeded a minimum value which for different conditions varied from 33 to 50 per cent of the coarse aggregate.

Sandstone as Coarse and Fine Aggregate in Concrete Pavement (Kentucky State Highway Department and University of Kentucky, Lexington)—The aggregate used on a 20-mile stretch of concrete pavement was a sandstone having a percentage of wear from 4 to 6, which was quarried, crushed, washed, and graded at a central plant. The material made excellent concrete, and the road, which was built as the result of extensive laboratory experiments conducted from 1923 to 1926, is standing up well. The data are being collected and compiled, but the results are not ready for publication.

Soundness Tests of Sandstone (Kentucky State Highway Department and University of Kentucky)—These were undertaken to determine whether the sodium sulfate soundness test can be depended upon when applied to sandstone, and whether stone failing to pass this test will make satisfactory concrete.

Effect of Grading on Void Content (National Sand and Gravel Association, Washington)—Two separate investigations were made, the first of which involved the study of the void content of combinations of 4 different sizes of gravel—¾ to 1½ in., ⅜ to ¾-in., No. 4 to ⅜-in., and No. 8 to No. 4. The 4 sizes were recombined to furnish a wide range of gradings, selected to give information for all possible combinations of the three coarser ones, and for typical combinations including the finer size. Tests were made on 57 different gradings, using two methods of filling the measure—"dry and loose" and "dry and rodded." Details of tests are available in mimeographed form; summary of more important conclusions published in *National Sand and Gravel Bulletin*, Oct., 1928; complete data to be published in pamphlet form.

The second investigation involved the study of void content of combinations of three sizes of sand—No. 14 to No. 4, No. 48 to No. 14, and 0 to No. 48. A wide range of gradings was selected to furnish information on all possible combinations of the three sizes used. Tests were made on 51 different gradings, using two methods of filling the measure—"dry and loose" and "dry and rodded." Further studies are contemplated with mixtures of the fine and coarse gradings used in the earlier investigations.

Effect of Pea Gravel on Strength of Concrete (National Sand and Gravel Association, Washington)—This is an investigation of the effect of various percentages of the finer sizes of gravel on the compressive and transverse strength and yield of concrete. Tests cover 8 different mixtures and 10 different percentages of gravel finer than ⅜ in., added to gravel graded to 1½ in. As these tests have been in progress only a short time, no results are available.

Special Investigations of Concrete Sands (National Sand and Gravel Association, Washington)—This study, which has just been started, will include a detailed investigation of sands exhibiting unusual characteristics when used in concrete and mortar.

Effect of Soft, Friable, Flat, and Elongated Particles in Gravel (National Sand and Gravel Association, Washington)—This investigation has just been undertaken on the effect of various percentages of soft, friable, flat, and elongated pieces of gravel on compressive and transverse strength, yield, surface hardness, and wear of concrete.

Effect of Grading of Gravel on Strength and Yield of Concrete (National Sand and Gravel Association, Washington)—Investigation of the effect of grading on properties of concrete, with particular reference to determining void content of aggregate consistent with high strength concrete. No results are yet available.

Study of Abrasion Tests of Fine Aggregates (U. S. Bureau of Public Roads, Washington)—Tests are under way to determine the most satisfactory method for making abrasion tests of sands.

Study of Tests for Quality of Fine Aggregates Which May Be Substituted for the Present "Strength Ratio" Test (U. S. Bureau of Public Roads, Washington)—Studies are being made to determine the possibility of utilizing the water-cement ratio strength law in the determination of the quality of concrete sands. Compression and cross-bending tests are being made with a large number of sands, using both wet and plastic mixes.

Comparison of Methods of Determining Moisture in Sands (Research Laboratory, Portland Cement Association, Chicago)—The following methods of determining moisture in sand were compared by testing in turn 3 gradings of sand—0-No. 4, 0-No. 8 and 0-No. 14, each sand being tested for 4 different percentages of moisture:

(1) Electric resistance moisture meter;
(2) Drying to constant weight in oven;
(3) Drying to constant weight with denatured alcohol;
(4) Displacement method using cylindrical container;
(5) Displacement method using A.S.T.M. flask;
(6) Specific gravity method using salt solution hydrometer.

A comparison of all methods used indicated that the best results were obtained with those methods requiring the simplest manipulation. For example, the two drying methods gave the best results, the manipulation consisting only of weighing before and after drying. The two displacement methods ranked next in point of accuracy, while the manipulation consisted of weighing the samples and also in determining the displacement of water. The hydrometer method, which ranked last in point of accuracy, required weighing the samples, measuring a definite quantity of salt solution, and in reading the hydrometer accurately. Details of each of these methods are given in the paper by W. R. Johnson at the 1929 Convention of the American Concrete Institute.

Investigation of Aggregates for Concrete (Research Laboratory, Portland Cement Association, Chicago)—A continuing investigation to study as fully as possible the role of the aggregate in the finished concrete, the arrangement of particles, their effect on strength, permeability, durability, and other properties of the concrete, and if possible to find any laws which might exist between the properties of the aggregate and those of the concrete. Thus far, a number of tests have been undertaken or are contemplated covering the following subjects:

(1) (*a*) Effect of gradation and size of coarse aggregate, and
(*b*) gradation of fine aggregate on workability and strength of concrete.
(2) Effect of gradation (fine-coarse ratio) of aggregates on density of concrete containing varying amounts of paste.
(3) Unit weights of different gradations of various aggregates.
(4) Tests of special aggregates.
(5) Study of variation of water content of concrete.
(6) Effect of shape and surface peculiarities of particles.
(7) Relation between strength of aggregates and strength of concrete.
(8) Relation between elastic properties of aggregate and concrete.

III. PLAIN CONCRETE

ABSORPTION

Absorption Tests of Mortar and Concrete (Reaseach Laboratory, Portland Cement Association, Chicago)—Two series of tests were made in which the following were studied:

(1) Quality of the concrete as affected by water-cement ratio and curing,
(2) Loss in weight of concrete on drying,
(3) Absorption of water by dried concrete,
(4) Rates of loss in weight and absorption,
(5) Temperature of drying preliminary to the absorption test, and
(6) Size and shape of specimen.

The specimens in both series were dried at each of two temperatures, 167 deg. F. and 239 deg. F. (75 and 115 deg. C.) in an electrically heated and controlled oven. Absorption tests were made by immersing the dried specimens in water at room temperature; after various immersion periods they were taken from the water and weighed, the excess water first being removed with a damp cloth. Results and conclusions of this investigation are given in a paper by Raymond Wilson before the 1929 Convention of the A.C.I. on "Limitations of the Absorption Test for Concrete Products."

ADMIXTURES

Effect of Diatomaceous Earth on the Modulus of Elasticity and the Strength of Concrete (University of California, Berkeley)—This investigation comprised tests of 6 by 12-in. concrete cylinders using diatomaceous earth to the extent of 3 per cent by weight of the cement. Three mixes were used, the richest being 1:3 and the leanest 1:7. For each mix three gradations of aggregates were used, the fineness modulus varying from 4.7 to 5.6. For each mix half the specimens were plain concrete without admixture. The slump in all cases was about 5 in., the consistency of the concrete being maintained constant by measurement with the flow table. All specimens were cured for 7 days in water and then stored in air until time of test. In general, compressive strength was increased through the use of diatomaceous earth for the lean mix regardless of age at time of test, but decreased somewhat by use of diatomaceous earth for the rich mix. There was no evidence that the modulus of elasticity was affected by the admixture.

Effect of Celite on Strength and Bulking of Concrete (Georgia School of Technology, Atlanta)—The purpose of this series of tests was to determine (1) the effect of celite on slag, gravel, and crushed rock concrete, and (2) the increase in volume of concrete in which celite was used. 6 x 12-in. cylinders for strength tests were made from a 1:2:4 mix by volume containing 3 lb. of celite per sack of cement. The specimens used in the bulking tests were 4 in. by 4 in. by 3 ft. There was a slight increase in strength at 28 days in concrete in which celite was used, and an increase of about 7 per cent in volume over that of concrete without celite.

Effect of Sugar on Strength of Concrete (Kentucky State Highway Department and University of Kentucky, Lexington)—Very small amounts of sugar reduced the strength to almost nothing. Samples of cement or sand sent to the laboratory in sugar sacks caused such trouble. Results of tests were published in *Kentucky Highways*, v. 3, September, 1928.

Effect of Dirty Sand and Clay on the Permeability and Durability of Concrete (Rensselaer Polytechnic Institute, Troy, N. Y.)—An investigation of the permeability and durability of concrete, in which dirty sand or sand containing varying amounts of clay will be used.

Effect of Kind and Amount of Lime on Volumetric Changes of Cement-Lime Mortars (University of California, Berkeley)—This investigation has just been begun and covers tests on the effect of amount of lime on cement-lime mortars when cast both in the form of bars and when used in brickwork.

Effect of Admixtures on Concrete (Delaware, Lackawanna, and Western Railroad Company, Hoboken, N. J.)—An investigation of various admixtures for the purpose of determining whether they increase workability without reducing strength. Specimens were made from concrete without and with

various percentages of admixtures, and tested at various ages for compressive strength. The results to date indicate that most admixtures decrease strength, the only exception thus far being trass. There is still a question, however, as to whether the increase in strength in the latter case was due to the admixture or to the reduction of mixing water. The 1-yr. tests on trass specimens showed a reduction in tensile strength.

AUTOGENOUS HEALING

Autogenous Healing of Mortar Briquets (University of Colorado, Boulder)—This extensive investigation in some cases included as many as four retests on more than 1000 standard tension briquets of various proportions of standard and natural sands, many different brands of cement, a wide range of water-cement ratios, a variety of ages at initial and subsequent tests, and various combinations of air and water curing. Specimens were cured and healed in both running and stagnant water without apparent effect upon either the curing or healing. Few general laws have been defined, but some rather striking results have been obtained. In all cases either a rubber band or a thread was used to hold the broken halves together. The results are summarized as follows:

(1) A neat cement briquet had a 3-day strength of 248 lb. per sq. in. Three months later it had a retest strength of 221 lb. In like manner a 1:3 standard sand briquet gave strengths of 247 and 143 lb. Normal 3 mo. initial strengths for these mixtures were 690 lb. for neat and 540 lb. for 1:3.

(2) Briquets tested initially at 3 mo. showed appreciable strength recoveries two weeks later. The highest of these were 33 lb. for neat cement and 4 lb. for 1:3 standard sand.

(3) Specimens broken and left separated over a radiator for 6 months showed healing when the broken ends were reunited and the specimens re-immersed.

(4) Specimens showed recovery after being initially broken after 2½ years of water curing.

(5) At the fifth breaking (fourth retest) 1:3 standard sand and neat cements, respectively, gave maximum strengths of 12 and 11 lb. The majority of specimens showed no recovery at the fifth breaking.

(6) In general, the strength recovery is less as the age of the specimen at initial test increases. It is usually less for succeeding tests. Although healing seems to occur for all cements, mixtures, sands, and water-cement ratios, it is usually more pronounced for the neat cement.

(7) Recovery can only occur in the presence of moisture. The healing period has an unevaluated effect upon the extent of recovery. Longer healing periods give greater recovery.

(8) Healing is from the inside out. Fresh crystal faces are often visible over the healed area. There is never healing about the outer margin of the break. There is no evidence of healing due to redeposition of soluble salts as has been previously suggested in connection with compressive autogenous healing. It seems that recovery is more probably due to continued curing; that is, a continuation or resumption of the normal curing process which continues for years under favorable conditions.

BOND

Bond between Concrete and Hollow Tile (National Bureau of Standards, Washington)—Tests were made to determine the effects of moisture content and water absorption of hollow tile on the strength in shear of the bond between

concrete and tile. The strengths were greatest when concrete mixtures containing the minimum amount of water necessary for their proper placement were used in combination with either dry or slightly dampened tiles of medium absorption. A complete report on the tests is given in "Bond between Concrete and Hollow Tile," by J. C. Oleinik, *Engineering and Contracting*, v. 67, January, 1928.

CHEMICAL ANALYSIS

Uniformity of Distribution of Cement in Concrete (Iowa State College, Ames)—A series of cores were drilled from a vertical slab which had been poured under known conditions using a known quantity of water, cement, sand, and coarse aggregate. These cylinders were broken down into the original constituents by Burke's method, and the distribution of materials throughout the slab compared. A similar process was followed with cores taken from actual highway pavements laid under supervision of the Iowa State Highway Commission. When this investigation is completed, the results will show the uniformity which may be reasonably expected in field construction of pavements.

CONCRETE PRODUCTS

Physical Properties of Cast Stone (National Bureau of Standards, Washington)—This study was undertaken with a view to establishing a federal specification for cast stone. A large number of samples from manufacturers throughout the United States were collected and tested. Specimens were cut or drilled from the material as it was received. Compression tests were made on 2 x 2-in. cylinders and the modulus of rupture of 1 x 1-in. prisms was determined using a 6-in. span. Rate of absorption and porosity tests were made on the broken halves of the modulus of rupture specimens. Weathering tests were made on all the specimens, and freezing and thawing tests on the 2 x 2-in. cylinders. Available results show that the compressive strength is high, 94 per cent of the specimens having a strength over 3000 lb. per sq. in. with an average value of 6200 lb. per sq. in.

CRAZING

Causes and Control of Hair Cracking in Concrete Pavement (Kentucky State Highway Department and University of Kentucky, Lexington)—Field conditions are being reproduced in the laboratory in an effort to determine the conditions that cause hair cracking, and to find a satisfactory method of prevention.

CURING

Curing Concrete by Use of Impervious Films of Asphalt and Other Materials (California Division of Highways, Sacramento)—An investigation of methods of curing by conserving the mixing water under a waterproof film applied to the surface of concrete pavement with a view to eliminating use of earth blankets and water. Tests were made on sections of highway under normal construction conditions. The sections were coated with various materials as were co-related concrete cylinders and slabs cast on the job. The slabs were removed at 10 and 25 days, sawed into beams and broken. Cores were also drilled from the same location in which the cylinders were cast. This study showed that concrete cured by the use of damp earth or by ponding gave greater strength than that cured in any other manner.

Efficacy of Sealing in Mixing Water as a Method of Curing Concrete (Minnesota Highway Department, Minneapolis)—A series of tests was made to determine whether concrete can be properly cured by preventing loss of mixing water, with an ultimate object of using bituminous-treated paper to cover fresh concrete pavements in lieu of other curing methods. Beams and cylinders were cast in regular forms, then sealed in asphalt paper containers, and tested

at 7, 14, and 28 days; strengths were compared with strengths of moist-cured specimens. Tests indicate that sealing in mixing water develops a high percentage of inherent strength, but the method is not so effective as moist curing. There was some question as to moisture content at the time of test, and additional work is planned for the purpose of checking this point.

Curing of Concrete Pavement (Tennessee Department of Highways, Nashville, in cooperation with the Bureau of Public Roads)—A study of the relative merits of twenty methods of curing on 17 miles of plain concrete pavement in southwestern Tennessee. One-half the pavement was cured by contractor and the other half was variously cured by the Division of Tests. Each of the 20 methods of curing was used on not less than 1000 ft. of pavement—about a day's run. 6 x 6 x 46-in. beams were broken in transverse test, 12 beams being made for each section of earth cure, 12 for research cure, 4 on blanks, and 4 on surface coatings with surface of beam sealed on 3 sides. No conclusions are available for publication, but a report will be published by the Tennessee Department of Highways and the Bureau of Public Roads as soon as the fourth, and final, cycle has been completed.

Comparison of Curing Methods of Concrete Paving (Georgia State Highway Department, East Point)—The purpose of this investigation was to determine the efficacy of various types of pavement curing. A nine-mile stretch of pavement was divided into three sections, each section being cured by a different method; namely, earth and water, sodium silicate, and surface application of calcium chloride. Cores were tested, a crack survey made, and a surface test made of each section at an age of 6 months. Results published in *Engineering News-Record* for August, 1928, showed little difference for the three types of curing.

Curing Concrete Pavement (Kentucky State Highway Department and University of Kentucky, Lexington)—This investigation consisted of a comparison of calcium chloride on the surface and one day wet burlap curing. The results to date indicate that 24-hour wet burlap will provide the necessary curing.

Effect of Curing Conditions and Condition at Test upon Compressive Strength (University of Colorado, Boulder)—About 400 6 x 12-in. compression specimens were tested in this investigation, and a number of 2 x 4-in. specimens were included for comparison. Proportions were 1:2:0, 1:2:3, 1:3:0, 1:3:4½, and each group of identical specimens was treated as follows:

1 stored in air and tested dry at 1½ years.
1 stored in air and tested wet at 1½ years.
1 stored in water and tested wet at 1½ years.
1 stored in water and tested dry (3 mo. in air) at 1½ years.
3 stored in water and tested wet at 7 and 28 days and 3 mo.

Water-cement ratios were varied over the workability range for each proportion. The same sand and cement were used throughout, but the gradings of aggregate used were 1:1½, ¼:1½, ⅜:½ for both crushed stone and gravel. Celite was used as an admixture in parts of the series. Specially devised instruments were used and loads were applied continuously to failure in obtaining both longitudinal and lateral stress-strain data. The following data were taken:

(1) Data for unit weight as made, capped, stored, tested, etc. This included water loss by leakage from mold between making and capping, and shrinkage during the same interval. Weight data include rates of evaporation and rates of absorption for all specimens dried in air or soaked prior to test.
(2) Ultimate compressive strength of all specimens tested at 7 and 28 days and 3 months.
(3) Both lateral and longitudinal stress-strain relation for all specimens, tested at 1½ yr., for modulus of elasticity and Poisson's ratio determinations.

(4) Autogenous healing data on some specimens.
(5) Comparisons between 2 x 4-in. and 6 x 12-in. specimens with reference to evaporation, absorption, strength, modulus of elasticity, and Poisson's ratio.

The conclusions may be summarized as follows, though only the more obvious results are offered due to the enormous mass of data which must be studied:

(1) For specimens cured alike, those that are air dry at test will be 20 to 30 per cent stronger than those wet at test.
(2) It may require several months to attain approximate air dryness; that is, about constant weight, but a 24-hr. immersion gives practical saturation.
(3) Air-cured specimens attained less than half the strength of those cured in water at the age of 1½ yr.
(4) If specimens are not in the same condition at test, the wet or dry state alters the strength so greatly that strength comparisons mean little.

In a later series on the same subject, eighty-four 6 x 12-in. specimens were made from concrete of the following mixes:

1:2:3; water-ratio 0.80; slump 6 to 8 in.
1:3:4½; water-ratio 1.10; slump 1 to 4 in.
1:4:6; water-ratio 1.20; slump 0.2 to 2.8 in.

The same materials were used for each group, including washed sand (0–4) and crushed quartzite (¾–1). Specimens were tested at 1, 3, 7, and 28 days, and 3 and 6 mo. for both standard and air curing. Stress-strain measurements were taken on all the specimens by special semi-autographic means, and other tests included evaporation, absorption, and Poisson's ratio. The results are not yet available for publication.

Effect of Steam Curing on Strength of Cement Mortars and Concretes (University of California, Berkeley)—The variables in this series of tests, which has just been started, will be steam temperature and period of steam curing.

Study of Methods of Curing Concrete (U. S. Bureau of Public Roads, Washington)—Studies of the behavior of a number of concrete slabs, 2 ft. wide and 200 ft. long, cured by various methods. The effect of various amounts of steel reinforcing, spacing of expansion joints, moisture and temperature changes, and subgrade resistance on the amount of cracking is being investigated, as well as the relative surface hardness of the various sections.

Study of Methods of Curing Concrete (Research Laboratory, Portland Cement Association, Chicago)—A study of the different methods of curing concrete controlling as carefully as possible such variable factors as temperature, humidity, moisture content of specimens at time of test, and any others that might influence the results.

DAMS

Tests on Stevenson Creek Dam (National Bureau of Standards, Washington)—The testing of the Stevenson Creek Dam, which was built by the Engineering Foundation in 1926, has been completed. This work was a part of the general investigation of methods for designing arch dams being made by a committee of the Foundation. The dam is a single-arch type, 60 ft. in height, with a radius of 100 ft. During construction, provisions were made for making measurements of the temperatures in the concrete, and the deformations and deflections of the dam produced by temperature and loads.

The measurements made during the tests were found to be adequate for checking the value of design formulas. None of the current methods of

design was found to be entirely adequate for representing the conditions in the dam, but the results of model tests agreed quite closely with the results of the dam tests. The results are published in the Report on Arch Dam Investigation, Proceedings of the American Society of Civil Engineers, Part 3, May, 1928; and in the paper by W. A. Slater, "Some Features of Testing of Stevenson Creek Arch Dam," *Proceedings* of the American Concrete Institute, v. 24, p. 273, 1928.

Investigations of Models of Arch Dams (University of Colorado, Boulder; U. S. Bureau of Reclamation; and Arch Dam Committee of Engineering Foundation)—Work has been completed on a $^1/_{12}$ size model of the Stevenson Creek experimental dam. Remarkable agreement was obtained, and extensive auxiliary tests are in progress including the following:

(1) Compression tests on 2 x 4-in., 3 x 6-in., and 6 x 12-in. cylinders for various ages and curing conditions. Semi-autographic records of stress-strain data, both lateral and longitudinal, taken in all cases. Modulus of elasticity and Poisson's ratio determinations made for all conditions and sizes of specimens. Weight data also included.

(2) Parallel strength tests with load deflection data are being taken on 3 x 3 x 40-in. and 4 x 6 x 38-in. beams.

(3) Long-time flexural flow tests at 25 and 50 per cent of the ultimate strength are being made on 3 x 3 x 40-in. beams. Some remain in air and some in water while loaded, and others are loaded after a period in air following moist curing. One beam on a 38-in. span was removed from water at 28 days and loaded with 50 per cent of its ultimate strength. Initial deflection was 0.0090 in., and after 2 months the total was 0.0445 in. The companion specimen tested to failure at 28 days reached a maximum deflection of 0.0195, which was less than half that attained from flow at half the ultimate load.

(4) Compressive strength of 4 x 14-in. specimens. Loads are 25 and 50 per cent of ultimate strength. Tests in water and air using car springs similar to tests of Prof. R. E. Davis. Various ages and conditions similar to flexural flow program.

(5) Tensile flow of 3 x 18-in. specimens in air at various ages and curing prior to loading. Fifty per cent of ultimate strength being applied through car springs.

(6) Tensile strength tests for ages and conditions paralleling other tests. 3 x 12-in. specimens used and longitudinal strains taken.

(7) Torsional strength tests, in which detrusions are measured and modulus of rigidity obtained.

(8) Volumetric changes due to curing and air drying are being measured. Beams 3 x 3 x 40-in. with end plugs used. These tests are similar to Prof. R. E. Davis' tests along similar lines.

All the tests are on a single concrete or mortar mixture, identical with that from which the model was poured. Proportions are 1:3.25 by weight, with a water-cement ratio of 1.00 and slump of 9.1 in. Local cement is used, and the aggregates are granite screenings (0–⅜) from Stevenson Creek, California.

Compressive control samples were taken from the concrete of the testing pit and that of the supplementary base for the model.

The above summary covers the major work completed or nearing completion. Much new equipment has been devised. Descriptions of some of this will doubtless appear soon in connection with a formal report on the tests. A new investigation on a model of the Gibson Dam will soon be launched with an accompanying auxiliary program of tests.

DURABILITY

Effect of Alkali on Concrete Drain Tile (Engineering Experiment Station, Iowa State College, Ames)—Two bulletins have been published covering the action of alkali on concrete drain tile and hydraulic cements. Bulletin 74 covers the chemical action of alkali on hydraulic cements, and Bulletin 89 the action of alkali solution on concrete drain tile. Both bulletins resulted from laboratory studies, the first being obtained by special chemical control, and the latter by observing the action on pieces of standard tile left in various solutions over a long period of time.

Durability of Concrete Exposed to the Weather (Washington University, St. Louis)—This investigation was undertaken to determine: (1) the effectiveness of certain waterproofing materials and various proportions of mix and curing conditions in making concrete resistant to capillary action and absorption under no appreciable pressure; (2) the effect of these added materials upon the strength of concrete; (3) the minimum depth of embedment of steel required to insure adequate protection when the above materials are used as waterproofers or water repellents; and (4) the effect of protective paints on both plain and deformed reinforced bars with respect to bond.

Action of Sulfate Ground Waters on Concrete Made from Lumnite Cement, Ciment Fondu, and Ciment Electrique (Atlas Lumnite Cement Company in cooperation with Bureau of Public Roads)—In the extensive investigation carried out by the Bureau of Public Roads on the effect of sulfate soils and waters on concretes made from various kinds of cements, studies were included of specimens made with Lumnite cement, Ciment Fondu, and Ciment Electrique, using a number of mixes and various curing conditions. Some of these specimens are under observation in the laboratory at University Farm, St. Paul, Minnesota, while others are in storage in the sulfate waters of Medicine Lake, South Dakota. From the available data it may be stated that when properly used, alumina cements offer greater resistance to the action of sulfate waters than do portland cements. A possible exception to the foregoing statement is portland cement concrete which has been subjected to long-time curing in water vapor at 212 deg. F. and higher. Concretes thus cured show little deterioration after exposure for as long as 6 years.

A complete description of the making, curing, and storage of many of these specimens, as well as the results obtained and the conclusions drawn, has been reported from time to time by D. G. Miller. Reports of some of this work may be found in various publications, including *Concrete*, April, 1926; *Public Roads*, October, 1925; *Public Roads*, November, 1927; *Proc.* A.S.T.M., 1928.

Investigation of Concrete Structures in Service (Research Laboratory, Portland Cement Association, Chicago)—This investigation has for its object a nation-wide survey and a thorough study of the behavior of concrete structures under service conditions in order to obtain information on the factors affecting the durability of concrete, as well as to develop laboratory tests which may be coordinated with field examinations. A study is being made of the behavior of various types of structures under different conditions of exposure, and the causes of such unsatisfactory behavior as may be encountered are being intensively investigated. To date over 200 structures practically all subjected to outdoor exposure have been examined.

Durability of Concrete (Research Laboratory, Portland Cement Association, Chicago)—The different factors affecting the durability of concrete structures are being investigated, and include compression, absorption, permeability, and freezing and thawing tests on 6 x 12-in. cylinders made according to A.S.T.M. Standard Methods. 6 x 6 x 32-in. prisms were cast in a vertical position in three layers and rodded according to the standard method, and 6 x 12½ x 32-in. specimens cast in a horizontal position, the concrete being placed in the center of the mold and the ends filled by allowing the concrete to flow from the center. For dry consistencies, the concrete was worked toward the corners with a

trowel, while for wet consistencies there was considerable segregation, the mortar flowing into the corners ahead of the coarse aggregate. The prisms were sawed into 6-in. cubes for freezing and thawing tests and 2-in. thick discs for permeability tests.

EXTENSIBILITY

Extensibility of Concrete (Purdue University, Lafayette, Ind.)—These tests were conducted to determine the toughness qualities of various concretes; that is, their ability to withstand deformations without the appearance of surface fissures. Since the permanence of concrete is largely dependent upon the preservation of the surfaces, the study of extensibility as well as means of increasing extensibility is of importance. The test specimens consisted of plain and mesh reinforced concrete beams tested both under slowly applied progressive loads and rapidly repeated or fatigue loads. The effect of other variables, such as the mix, curing, exposure, age, and kind of aggregate has also been observed. Results will be published in a bulleting of Purdue University entitled "Physical and Mechanical Properties of Portland Cements and Concretes."

FATIGUE

Fatigue of Mortar and Concrete (Purdue University, Lafayette, Ind.)—This investigation has been continued from time to time over a period of about 5 yr. and brief reports have appeared in several publications. Some of the later results will be published in a bulletin on "Physical and Mechanical Properties of Portland Cements and Concretes," the discussion including not only a detailed account of the Purdue tests, but also a complete summary of the more important investigations on fatigue of concrete that have been conducted throughout the country in recent years.

FIELD CONTROL

Quality Concrete Projects (Wisconsin Highway Commission, Madison)—This study was made to determine the design and methods necessary to construct concrete pavement of maximum strengths with specific aggregates, and to obtain a given strength the most economically. The mixes were designed and the materials accurately controlled for the concrete on two paving projects about 6 miles in length. Beams, cylinders, and cores were taken, the specimens being made on the job. The beams were broken in the field at ages of 7 and 28 days, while the other specimens were tested in the laboratory.

FLOW UNDER LOAD

Flow of Concrete under Sustained Compressive Stress (University of California, Berkeley)—This is a new series of tests on a phase of concrete research which has been investigated several times before. In these tests the cylinders are continually under a stress of 750 lb. per sq. in., the mix being constant and the character of the aggregate varying. The aggregates include quartz, granite, limestone, sandstone, and basalt. The effect of moisture conditions, including variations in humidity, on the flow of concrete under sustained compressive stress will also be studied. Tests are in progress to determine the lateral as well as the axial flow in unreinforced concrete columns and in columns reinforced with longitudinal bars and spiral hooping. The axial deformations are being measured with strain gages, as in previous flow tests, but the transverse deformations are observed with a special piece of apparatus utilizing micrometer microscopes.

A supplementary investigation is contemplated in which the loads will be released periodically and then reapplied.

FREEZING AND THAWING

Study of Effect of Type of Coarse Aggregate on Resistance of Concrete to Repeated Frost Action (U. S. Bureau of Public Roads, Washington)—Concrete specimens containing a wide variety of coarse aggregates, most of which were questioned from the standpoint of durability, are being subjected to alternate freezing and thawing. Periodic examinations of the concrete are made for evidence of failure through frost action. Concrete is also tested for flexural strength in comparison with similar specimens which have not been frozen.

Tests of Concrete and Mortar Frozen at Different Periods within 24 Hr. after Molding (Research Laboratory, Portland Cement Association, Chicago)—An investigation of concrete structures in service has shown the need of information on the behavior of concrete which has been frozen during the early setting periods and later thawed out. In this investigation one group of tests consisted of placing 3 x 6-in. concrete cylinders in the freezing room immediately after making and removing them after they were frozen for periods of 1, 2, 5, and 8 hr., 3. 7, and 28 days, and 6 mo. In another group the specimens were frozen for various periods, 3 and 6 hr. after molding. In a third group specimens which were placed in the freezing room 1, 2, 4, 7, and 24 hr. after molding were frozen until 6 hr. before test. The temperature of freezing room ranged from 15 to 30 deg. F. Wherever possible compression tests were made on the specimens at ages of 7 and 28 days.

Freezing and Thawing Tests on Neat Cement, Mortar, and Concrete Specimens (Research Laboratory, Portland Cement Association, Chicago)—Specimens frozen at a temperature of about 15 deg. F. in a specially constructed room and thawed out in water at a temperature of about 120 deg. F. The specimens are placed in a container filled with water during the freezing period and no drying out is allowed at any time. In one group of tests, freezing and thawing tests are being carried out on neat cement, mortar, and concrete specimens of different water-cement ratios in a general study of the factors affecting the durability of concrete. The specimens are 6 x 12-in. cylinders and 6-in. cubes. About 30 cycles of freezing and thawings have been completed to date. In a second group, freezing and thawing tests are being made on 4½ x 1-in. mortar discs of different water-cement ratios cured under different conditions. About 100 cycles have been completed. Other miscellaneous freezing and thawing tests are under way.

JIGGING

Effect of Jigging during Setting Period on the Compressive Strength and Other Properties of Concrete (University of California, Berkeley)—These tests were made to determine the effect of jigging on the compressive strength of 6 by 12-in. concrete cylinders, the shrinkage of 3 x 3 x 40-in. bars, and the modulus of elasticity as determined by strain measurements on the 6 x 12-in. concrete cylinders. The duration of jigging period varied from one-half to one hour, and the rate of jigging varied from 40 to 150 vibrations per minute. Tests were made at the age of 28 and 90 days. The gradation of aggregate approached Fuller's curve, but with an excess of fines. It was found that the increase in compressive strength produced by jigging at the lower rates was marked, but at the higher rate the strength, in general, was less than that of the corresponding unjigged specimen. It appears that the increase in strength due to jigging is more marked at the later ages than at the early ones. The modulus of elasticity increased with the strength.

MODULUS OF ELASTICITY AND POISSON'S RATIO

Modulus of Elasticity and Poisson's Ratio for Plain Concrete (University of California, Berkeley)—During the past year a program of tests on 6 x 12-in. crushed stone concrete cylinders, which had extended over a period of four years, was completed. These tests were made to determine the effect of age and richness of mix upon the elastic properties of concrete, including the

modulus of elasticity and Poisson's ratio. Both axial and transverse deformations were measured with mirror extensometers for stresses up to 2800 lb. per sq. in. All specimens were tested to failure. The storage was in damp sand up to the time of test. The mixes were 1:3.5, 1:4.5 and 1:6. The results may be summarized as follows: (1) For a given concrete the modulus of elasticity increases rapidly at the early ages, but after two years the increase is small. That is, if the secant modulus of elasticity at 1000 lb. per sq. in. is 2 million lb. per sq. in. at one month, it may reach a value of 5 million lb. per sq. in. at three years; (2) The modulus of elasticity for a lean concrete appears to increase at a less rapid rate at later ages than it does for a rich concrete; (3) The stress-strain curve becomes straighter as the age increases; (4) Poisson's ratio increases with age and varies somewhat with the stress, but apparently is little affected by richness of mix. For the later ages Poisson's ratio ranges from about 0.17 to 0.20. It appears to have a minimum value of about 1000 lb. per sq. in., and to increase at both higher and lower stresses.

A new series of tests is now under way on specimens stored in damp sand, for the purpose of determining the effect of age and repeated load on the modulus of elasticity in Poisson's ratio.

Elastic Properties of High Alumina Cement (University of California, Berkeley)—These tests were made on 6 x 12-in. cylinders of high alumina cement concretes at ages varying from one day to six months. Two mixes were used, and the specimens were stored under water until the time of test The stress-strain relations were determined by compressometer measurements. In general the maximum strength was attained at the age of one week, after which there was considerable retrogression. The modulus of elasticity appeared to remain fairly constant regardless of age, the secant modulus of 1000 lb. per sq. in. being about 2½ million lb. per sq. in.

PAVEMENTS

Advantages of Quick-Hardening Cement in Pavement Construction and Maintenance (Pennsylvania Department of Highways, Harrisburg)—This project was undertaken to promote methods of construction or use of materials which might reduce inconvenience and traffic delays in congested areas. It was decided that if the customary curing period for concrete pavements and bases could be decreased by the use of special cements, or by increased cement content and a decrease of the water content, tests would indicate which method gave the best service and was the most economic. A comparison was made of the strength results of mortars and concrete using normal mixes, rich mixes with low water content, and super or special cements. Though observations were made of the durability of concrete under traffic, final conclusions cannot be drawn until long-time strength tests, abrasion tests, and flexural tests have been made. Some of the early results were published in the *Proceedings* of the Highway Research Board, v. 4, p. 145, 1926, and in *Engineers and Engineering*, v. 45, No. 5.

Design of Expansion Joint Spacing (Tennessee Highway Department, Nashville)—An investigation to determine the correct spacing for pre-molded expansion joint and to develop a formula by which it may be designed. Expansion joints of different widths were placed at different spacings in one continuous strip of concrete highway. Expansion and contraction movements were carefully observed by strain gage determinations, and the general condition of pavement observed during periodic crack surveys. This test road was augmented by determinations throughout the state for coefficient of friction between concrete slab and different soils and soil conditions. Conclusions available at this time indicate that expansion joint spacing should be designed with relation to the temperature of the concrete at time of placing, expected temperature of the locality, and coefficient of friction of subgrade soil.

Effect of Finishing Operations on Surface Strengths of Concrete Pavements (Minnesota Highway Department, Minneapolis)—Besides determining the

effect of finishing operations on concrete pavements, this investigation included a study of their relation to scaling. Mortar was extracted from the surface of the pavement, and tested for water content and compressive strength. It was found that each finishing unit brought water to the surface and reduced the surface strength. As there was a wide variation in the amount of finishing on different jobs and on different sections of the same job, the strength of the mortar decreased from 30 to 50 per cent as a result of finishing operations. Observations of any scaling will be made later.

Investigation of Expansion Joint Filler (California Division of Highways, Sacramento)—A study of the action of slabs which makes expansion joints necessary, and materials which are satisfactory as joint fillers. Joints were installed and inspected at various ages. Previous to 1922 no expansion joints were used; from 1922 to 1924 joints were 2 inches wide at 100-ft. intervals, and filled with a mixture of asphalt and sawdust; from 1925 to 1926 joints of redwood or elastite were used spaced 50 ft. apart; in 1927 1½-in. joints of premolded cork and asphalt were used spaced at 60-ft. intervals; in 1928 ½-in. joints were used of sponge rubber spaced at 60-ft. intervals with 20-ft. dummy; and in 1929 joints were ½-in. sponge rubber about 3 in. deep with slabs arranged to expand laterally from center to panel.

PERMEABILITY

Permeability of Mortar and Concrete (Research Laboratory, Portland Cement Association, Chicago)—This is a new investigation which has for its purpose a comprehensive study of the factors which affect the watertightness of concretes and mortars. Permeability apparatus for measuring the amount of water penetrating mortar and concrete discs under constant pressure has been developed and with the present arrangement 44 specimens may be tested simultaneously. Tests are under way on concrete specimens of various water-cement ratios cut from different sections of concrete cylinders and on a number of mortar and concrete mixes using ordinary portland and special cements. A few concrete specimens containing various admixtures have been tested. The results to date show that the method of early curing is a very important factor in obtaining impermeable concrete.

PLACING

Variations in Strength of Concrete Cylinders Due to Rodding (Kentucky State Highway Department and University of Kentucky, Lexington)—Test cylinders were placed in various ways. It was found that with reasonable care in rodding, tamping, or puddling the concrete, good results were obtained. The size of the rod and the amount of tamping after a reasonable time did not have any noticeable effect.

PROPORTIONING

Importance of Water Control in Concrete (Pennsylvania Department of Highways, Harrisburg)—To stimulate the importance of careful control of concrete mixtures on construction projects, a portable testing machine was placed on each contract and the water content regulated to secure maximum results within the limits of workability. Grading and proportioning of the aggregates were carefully controlled, compensating for the bulking in fine aggregate in volume and weight proportioning. Comparisons were made of the relative strengths of the cement and concrete, the water having been carefully controlled to secure maximum strengths consistent with workability. The water content was determined on the various contracts and the actual strength compared with the expected strength for the given water content. Close checks were obtained between the expected and actual strengths, and in general, the strengths were higher than had been previously obtained. An important result was that the field forces became interested in quality concrete.

Scientific Methods for Proportioning Concrete Mixtures (Tennessee Highway Department, Nashville)—This investigation involved determination of the following: (1) the relation between the water-cement ratio and the modulus of rupture for each type of coarse aggregate available in Tennessee; (2) the relations between different cements and sands available for Tennessee construction. Each portland cement used in Tennessee was combined with each sand in predetermined absolute proportions varying from 1.1 to 2.4 by 0.1 proportion. As the percentage of sand was increased from minimum to maximum, a peak point was determined; (3) the workability of combinations of each coarse and fine aggregate, cement, and water as used in their absolute volume proportions under the fineness modulus method of proportioning concrete. This study was made in cooperation with and after the manner of the New Hampshire Highway Department. A watt-hour determination was derived for all absolute volume proportions which worked satisfactorily. The watt-hour determination was set up as a standard of workability; (4) the design of mixtures using materials submitted by contractors. From the modulus of rupture required by the specifications and the coarse aggregate submitted by the contractor, an absolute volume proportion of sand-cement ratio is taken from the peak of the sand-cement curve. The absolute volume proportion of sand, cement, and water-cement ratio volume of water are mixed into a mortar and placed in a tub-mixer. Coarse aggregate is added to the extent of the workability factor derived from the tests in (3) above and the absolute volume proportion of sand, cement, and coarse aggregate and the water-cement ratio are given to the field forces as a criterion for the proportions of materials.

Method of Design of Concrete Based on Cement Factor, Strength, Water-Cement Ratio and Voids in Fine and Coarse Aggregate (Kentucky State Highway Department and University of Kentucky, Lexington)—Three factors are being considered in this investigation: (1) control of water, (2) control of cement factor or the solid volume of the aggregate, and (3) control of fine aggregates as a function of the voids in coarse aggregate.

PROTECTIVE TREATMENTS

Protective Treatments for Concrete Silos (Lehigh Portland Cement Co., Allentown, Pa.)—The opportunity for investigating this subject arose in the summer of 1927 in connection with the erection of two large reinforced concrete silos by a distributor of dairy products. Immediately after completion, the silos were filled, and when emptied the following spring the inner wall surfaces were found to have been etched by the ensilage liquor. While the condition was not alarming, the opportunity was taken to treat wall sections of the year-old silos, as well as the walls of two new silos, with a number of experimental coatings, including cement, paraffin, and bituminous treatments. Small concrete cylinders of varying cement content and degree of curing were also stored in one of the silos to determine the effect of richness of mix and age of concrete when exposed to the action of ensilage liquors.

STRENGTH—COMPRESSIVE

Failure of Concrete under Combined Compressive Stresses (T. & A. M. Department, University of Illinois, Urbana)—Tests of concrete were made under compressive stresses applied by hydraulic pressures in one, two, or three directions to study the phenomena of breakdown of the material. Studies of spirally reinforced concrete which was subjected to combined compression were also made. Results of these tests published in Bulletin 185 of the University of Illinois Engineering Experiment Station, November, 1928.

Study of the Technique of Making and Testing the Standard Concrete Compression Cylinder (T. & A. M. Department, University of Illinois, Urbana)—Study was made of the effect of various methods of mixing concrete, molding, capping, storing, and testing cylinders.

Relation between Strength and Elasticity of Concrete in Tension and Compression (Iowa State College, Ames)—Bulletin 90 published recently covers the laboratory work on this subject and gives a considerable quantity of data on the behavior of concrete specimens in compression and tension.

Effect of Premixing Cement and Water (Barney-Ahlers Construction Co., New York City)—In this study a comparison was made of the yield and strength of concrete (1) using standard methods of proportioning and (2) premixing the cement and water. Results to date show a marked improvement in the workability of concrete made from premixed cement and water.

STRENGTH—FLEXURAL

Effect of Repeated Stresses on Plain and Reinforced Concrete (Cornell University, Ithaca, N. Y.)—In making this study of the effect of repeated stresses, particular attention was given to the effect of different coarse aggregates on resistance to repeated loads. Preliminary tests using gravel aggregates were made on beams 6 x 6 x 50-in. Loading was at the third-points and was accomplished by application of force at one end rather than in the center as is customary in third-point loading. The beams were alternately bent upward and downward and readings of deformations and progress in deformation in extreme fiber were taken. Companion test pieces were made in compression, transverse static, static tension, and repeated tension.

Tests of Concrete Beams Using Various Aggregates (North Carolina State Highway Commission, Raleigh)—These tests were carried out to determine the flexural strength of beams using standard crushed stone aggregate and pebble coarse aggregate of varying qualities. To date a comparatively poor quality pebble has been compared with a standard crushed stone. Beams using pebbles and varying in depth, and crushed stone of constant depth and varying mixes and test age were broken by the cantilever method. Beams with pebble aggregate showed 25 to 30 per cent wear under a modified abrasion test, and the conclusions are that beams from this coarse aggregate should be approximately 1 in. deeper than the standard coarse aggregate beams to give equal strength.

Relation of Coarse Aggregate to Transverse-Compression Ratio (Tennessee Highway Department, Nashville)—The purpose of this investigation was to obtain data on the relation of compressive to transverse strength of concrete made from different coarse aggregates.

Using each type of coarse aggregate in individual mixes, concrete cylinders and beams were made of the same materials and proportions. Each type of coarse aggregate used in Tennessee was submitted for use in this investigation. The results show that there is a difference in the compression-transverse factors when different coarse aggregates are considered.

Study of Effect of Character of Sand Grains on Flexural Strength of Cement Mortars (U. S. Bureau of Public Roads, Washington)—Sands of widely varying mineral composition are being tested as mortars in flexure. Every factor except the character of the sand grains has been eliminated in these tests.

Study of Effect of Type and Gradation of Coarse Aggregate on the Flexural and Tensile Strength of Plain Concrete (U. S. Bureau of Public Roads, Washington)—This is a general investigation the purpose of which is to determine the effect of type, grading, and amount of coarse aggregate on the flexural strength of concrete, all other variables being eliminated. Seven crushed rocks, 7 gravels, and 3 blast furnace slags are being investigated.

TEMPERATURE EFFECTS

Effect of Temperature upon Strength of Cement Mortars and Concretes (University of California, Berkeley)—A preliminary series of tests on 6 by 12-in. concrete cylinders, 2 x 4-in. mortar cylinders, and standard mortar briquets was completed. Several mixes were used and all specimens were stored in

water until tested. The temperature varied from 36 deg. to 140 deg. F., and the age at time of test varied from 7 to 135 days. In every case it was found that the strength in both tension and compression decreased as the temperature increased, this decrease being in some instances as much as 25 per cent less at the highest temperature than at the lowest. Apparently the effect was more marked for rich mixes than for lean ones.

A supplementary series of tests was just begun for the purpose of determining the effect of temperature at time of test on the strength, modulus of elasticity, and Poisson's ratio. These tests are being made on 6 x 12-in. concrete cylinders of several mixes and aggregates, part of the specimens being air-dry and part water-soaked. Temperatures will vary from 10 to 140 deg. F.

Effect of Frost on Concrete Road Slabs (Colorado State Agricultural College, Fort Collins)—The purpose of this investigation was to measure the heaving due to frost on 6-in. concrete road slabs 20 ft. square, one placed on dry, well-drained ground, and the other on wet, poorly-drained soil. Measurements were taken with a micrometer gage of the rise and fall of these slabs as the weather varied throughout the year. Records were made of frost penetration and condition of subgrade from time to time. Movement of the slabs varies directly with the temperature.

Fire Resistance of Concrete Masonry Units (Research Laboratory, Portland Cement Association, Chicago)—A series of fire tests on twenty 8 x 8 x 16-in. concrete block panels covering a study of the effect of grading of aggregate, cement content, and consistency on fire resistance of concrete. The primary purpose of these tests is to secure data on which to base recommendations for the manufacture of sand and gravel masonry units having a maximum fire resistance.

TESTS AND TEST METHODS

Effect of End Conditions of Concrete Cylinders on Compressive Strength and Observed Deformations (University of California, Berkeley)—This investigation was made to determine the effect of end irregularities upon the compressive strength and also upon the axial and transverse strains as determined by measurements with extensometers bearing against the outer surface of the cylinder. The tests were made at ages of 1 and 5 months. One-third of the specimens had the ends ground to a truly plane surface and were loaded over the entire end area, one-third were concentrically loaded through a disc at each end 3 in. in diameter, and the remainder were loaded through a 1-in. annular ring having a 6-in. diameter. It was found that there was a noticeable difference both in the axial and in the transverse deformations under a given load depending upon the manner of applying the load. Specimens loaded with the ring showed the largest strains, those loaded through the disc exhibited the least.

Effect of Size of Test Cylinder on Compressive Strength of Concrete (University of California, Berkeley)—These tests have just been started on cylinders ranging in size from 2 x 4 to 12 x 24-in. Part of the cylinders of each size are coated with asphalt as soon as they are removed from the forms, and then are stored in water until time of test; the remainder are cured in water for 7 days and then stored in dry air until test.

Value of Sand Bearing in Testing Compression Specimens Having Irregular Ends (North Carolina State Highway Commission, Raleigh)—This investigation was started with a view to eliminating the capping of cores from roads, broken beams, etc., when testing in compression. Laboratory and road-drilled specimens were divided into two groups, one group being tested by capping and the other using sand bearing. There is no advantage in using sand bearing when cap can be placed at the time specimen is made, or when specimen has ends regular enough to require only thin layer of capping material. There is, however, a considerable saving in labor and time when testing specimens having comparatively irregular ends.

Relationship between End Conditions of Concrete Cylinders and Their Strength (Kentucky State Highway Department and University of Kentucky, Lexington)—Cylinders were made having concave, convex, troweled, and perfect ends. Comparisons were made of capped and uncapped cylinders, of cylinders capped after 14 days, and cylinders tested with sand bearing. It was found from these experiments that if the true value of the concrete is to be obtained, there must be a perfect bearing on each end of the cylinder. Capping, except when made as an integral part of the specimen, cannot be relied upon. Sand bearing is very unreliable, and its value depends upon the condition of the cylinder end. Results of this research are published in *Kentucky Highways*, v. 3, August, 1928, and in *Highway Engineer and Contractor*, v. 19, October, 1928.

Effect of Capping on Concrete Cylinders (Colorado State Agricultural College, Fort Collins)—The purpose of these tests was to compare the strength of 6-inch concrete cylinders capped with different materials, which included Lumnite cement, plaster of paris, neat portland cement, and no capping. About 100 specimens were used for each type of capping, all the cylinders having been made in the same way and from the same mix. Cylinders show a higher strength when capped with Lumnite cement.

TIME OF MIXING

Time of Mixing Pavement Concrete (Georgia State Highway Department, East Point)—This investigation was made with a view to determining the effect of varying the time of mixing concrete. Only preliminary work has been done to date, the quality of the concrete being checked by cylinder, beam, and core tests.

Effect of Time of Mixing on Strength of Concrete (Pennsylvania Department of Highways, Harrisburg)—The purpose of this series of tests was to determine whether with improved mixing equipment the time of mixing could be decreased without reducing strength, consistency, uniformity, or workability. Two series of test beams and cylinders were molded from concrete mixed 30, 45, 60, 75, 90, and 120 seconds. Since it was believed that puddling would defeat the purpose of the experiment, the concrete was placed in molds without puddling, only the normal amount of handling to which concrete is subjected being used in placing. Short-time tests have been completed, and the results to date show that the 30 and 45-sec. concrete lacked uniformity, and required extra manipulation to place.

VOLUME CHANGES

Changes of Volume of Cements and Concretes (Purdue University, Lafayette, Ind.)—This project has extended over a period of more than 3½ yr., and has involved approximately 6000 volume change observations and about 3000 weight determinations. The specimens consisted of beams made from both neat cements and concretes in several mixes. The effect of other variables, such as the kind of coarse aggregate, gradation, etc. was also observed. Results of these tests will be included in a bulletin by the Purdue University Engineering Experiment Station under the title, "Physical and Mechanical Properties of Portland Cements and Concretes."

Coefficients of Expansion of Concrete (University of California, Berkeley)—These tests were made to determine the variations in the coefficient of thermal expansion of concrete in which the variables were water-cement ratio, richness of mix, type of aggregate, curing condition, and age of specimen. The coefficient of thermal expansion was determined for temperatures varying from 40 to 130 deg. F. In general, the coefficient of thermal expansion was lower for dry than for wet specimens, for high water-ratio than for low water-ratio, and at later ages than at earlier ones. The average coefficient of thermal expansion was 0.0000047 per deg. F.

Supplementary tests to determine the effect of repeated extremes of temperature on thermal expansion. Tests have just been started on 3 x 3 x 40-in.

concrete bars, the type of aggregate, water-cement ratio, and storage conditions being the variables. The aggregates include gravel, granite, limestone, sandstone, basalt, and quartz.

Additional tests were made to determine the volume changes in alumina cement concrete due to variations in moisture conditions, and to determine the thermal coefficient of expansion. The temperature tests indicate that richness of mix and age have little effect upon the thermal coefficient of expansion, the value of which was found to be 0.0000053 per deg. F. The moisture tests are still in progress.

Volumetric Changes in Concrete (University of California, Berkeley)—Tests are being made on gravel concrete, the variables being gradation of aggregate, richness of mix, and moisture conditions. Some of the data are included in the paper by R. E. Davis on "Volumetric Changes in Portland Cement Mortars and Concretes Due to Changes Other than Variations in Temperature." *Proc.* International Congress Testing Materials, 1927.

In one group of tests in this general investigation the specimens were 3 x 3 x 40-in. bars of concrete of three water-cement ratios and two mixes. Half the bars were stored in air at 70 deg. F., in 50 per cent relative humidity, and the remainder were stored in water. The shrinkage of the former and the expansion of the latter were observed periodically. The tests indicate that within certain ranges, the water-ratio has a considerable influence on the volume changes in mortar, the shrinkage during the period of drying being greater for the mortar with high water-ratio, and the swelling during period of storage in water being somewhat greater for the mortar with low water-cement ratio.

In another group of tests the aggregate used is entirely granite, and only one mix is used. Part of the specimens are stored continuously in water, part are stored in air at 50 per cent relative humidity, and part are subjected alternately to water soaking and air drying for periods of time varying from five days to three months.

Tests are now under way to determine the effect of various admixtures. such as lime and clay, on volume changes in cement mortars. The specimens are mortar bars and brick piers. Certain brands of modified cements are included.

WATERPROOFING

Investigation of Integral Waterproofing Compounds for Concrete (National Bureau of Standards, Washington)—This study was started in July, 1928, to determine the relative merits of various integral waterproofing compounds. Preliminary work was done on several mixes of concrete to establish a standard mix for use in this investigation. A 4 x 7-in. sheet metal cylinder was filled with concrete and clamped between two flanges. The amount of water penetrating the specimen was collected and measured at intervals. As the work is still in a preliminary state, no conclusions have been drawn.

Surface Waterproofing Materials (Hydro-Electric Power Commission of Ontario, Toronto)—An investigation of different methods and materials used in treating the exposed surfaces of concrete to waterproof and preserve them. 2 x 4-in. mortar cylinders and 6 x 3-in. wafers of concrete treated and untreated have been exposed for about a year, and are being examined and tested for absorption at stated intervals.

WEAR

Resistance of Floor Finish to Wear (Barney-Ahlers Construction Co., New York City)—Nine types of floor finish were ground with a rotating machine. Methods of testing and test results given in a paper before the 1929 A. C. I. Convention.

Relationship of Cement Factor to Wear and Strength (Kentucky State Highway Department and University of Kentucky, Lexington)—A special design of the

concrete mix used in which the fine aggregate was 1.2 times the voids in the coarse aggregate, and the cement factor 1.5 to 1.7. Comparisons were made of two limestones, one gravel, and two sandstone coarse aggregates in beams, cylinders, and 9-in. balls. When satisfactory aggregates of standard gradation are used a cement factor of 1.5 will produce concrete satisfactory for pavements.

IV. REINFORCED CONCRETE

ARCHES

Behavior of Reinforced Concrete Arch (Georgia Highway Department, East Point)—A study of the initial behavior and stresses in a reinforced concrete arch, including temperatures and rise and fall of arch rib. An open spandrel arch was constructed with a 160-ft. span and a 45-ft. rise. Electrical strain gages or telemeters were installed on steel before concrete was placed, as well as in the concrete while being placed. From the record of strains during setting time and while the deck was placed, temperature measurements, the rise and fall of rib, and the movement of abutments, it was concluded that there is initial stress in both concrete and steel due to temperature rise during setting and the difference in coefficient of expansion of steel and concrete, and that the fall in temperature rather than the rise and fall should be considered in temperature stresses in arch ribs.

BEAMS

Tests of Reinforced Concrete Beams (Lehigh University, Bethlehem, Pa.)—These tests are being made to determine the effect of variation in spacing of vertical stirrups on the strength of beams. The beams are I-shaped, 18 in. deep, with a 9½-ft. span, and the stirrup spacing varies from about 3 to 15 inches.

Flexural Tests of Reinforced Concrete Beams (University of Colorado, Boulder)—Twenty-three moist-cured 4 x 9-in. reinforced concrete beams with an effective depth of 8 in. were tested on a 7-ft. span at 31 days. The concrete mix was 1:2:3½ by loose volume with a water-ratio of 1.00. Various anchorages were used, stirrups were used in some beams, and the actual steel ratios were 0.0048, 0.0115, and 0.0375. Comparisons were made between plain concrete, under-reinforced, approximately balanced, and over-reinforced beams. Auxiliary compression tests on 6 x 12-in. specimens and bond pullout tests were made. Retests were made on the pieces to determine the strength relations for shorter spans. Greased rods with nuts and plates were used in some beams, and deflection data were taken at initial tests on all the specimens. Failures were obtained in bond, tension, diagonal tension, and compression. The range of beam strengths, as indicated by center load, was 1500 lb. for plain concrete, 2500 lb. for under-reinforced, 4500 lb. for balanced, and 16,000 lb. for over-reinforced. The indication is that there may be actual economy in the use of highly over-reinforced beams if proper means are employed to safeguard against bond and diagonal tension failures. The strongest beams (16,000 lb. center load) failed in compression as special anchorage and heavy web reinforcement were provided. Compression failures are rare in the literature of concrete beam tests. The test results have not yet been worked up, but the indications seem to point to a need for more tests along these lines.

Investigation of Beams with Compressive Steel under Continued Load (Northwestern University, Evanston, Ill.)—The purpose of this investigation was to determine the effect of plastic flow and shrinkage on distribution of total compressive flange stress between steel and concrete, and to determine the effect of various lengths of compressive steel on reduction of deflections. Two sets of five beams were used in this study, one set having been made from high-strength concrete and the other from low-strength concrete. Load was applied at center by a multiplying leverage device. No attempt was made to separate

shrinkage effect from time effect. The conclusions drawn to date are that compressive steel stresses in less than four months are more than double any stress based on theory, not taking flow into account, and that deflections are greatly decreased by compressive steel.

BOND

Studies of Bond in Reinforced Concrete Beams (T. & A. M. Department, University of Illinois, Urbana)—This investigation was undertaken to study the effect of continued loading at various intensities of load upon bond-slip relations. Beams were loaded for a long period by means of jacks and heavy helical springs, and slips were measured at regular intervals.

BRIDGES

Tests of the Arlington Memorial Bridge (National Bureau of Standards, Washington, in cooperation with the Arlington Memorial Bridge Commission)—This investigation covered a study of the temperatures and strains developed in one of the concrete ribs of the Arlington Memorial Bridge, Washington, D. C. The purpose of these tests was (1) to check the results of model tests, (2) to obtain data on the concrete rib, and (3) to obtain data on inelastic deformations due to shrinkage, flow, and changes in temperature.

Analysis of a Celluloid Model of the Yadkin River Bridge (Johns Hopkins University, Baltimore, in cooperation with the Bureau of Public Roads)—The object of these tests was to check the stresses measured in a reinforced concrete bridge over the Yadkin River by the Beggs deformeter method using a celluloid model. Influence lines were produced for moment, shear, and thrust at various sections of the arch rib, and from these influence diagrams measured stresses and deflections were checked. Results will be published as soon as the report is released by the Advisory Committee of the Yadkin River Project and the Research Committee of the Bureau of Public Roads.

REINFORCING STEEL

Arrangement and Placing of Reinforcing Steel (California Division of Highways, Sacramento)—This study was made to determine the proper mesh size to be used in practical construction. Various arrangements of reinforcement were installed in the field, including several different types of bar and mesh arrangement as to spacing and support in concrete pavements. Some of the pavements are on original good ground, some on defective ground, some on old concrete pavement, and others placed adjacent to old pavements as widening. It was found that each case required a different type of reinforcement. Conclusions from available data indicate that openings at least 12 x 14 are desirable in mesh or bar reinforcement to avoid having mats trampled out of position. Mats must be stiff enough to support themselves between points of support.

Investigation of Hooks as Anchorage in Reinforced Concrete (Washington University, St. Louis)—This investigation has just been started, and no results are yet available. Plain and deformed rods of varying size are being used in concretes of various compressive strengths, and only hooks which permit of practical fabrication are included. It is proposed to determine: (1) the effect of varying strength of concrete; (2) the effect of size of rod; and (3) the effect of shape of hook, semi-circular hooks to be compared with hooks of changing radius.

V. SUGGESTED RESEARCHES ON CONCRETE AND RELATED SUBJECTS

CEMENT

(1) Time of set of various cements for use in concrete placed under water.
(2) A new standard strength test for cement.

AGGREGATE

(1) Effect of arrangement of aggregate particles in symmetrical placement.
(2) Development of satisfactory field methods other than drying to obtain percentage of moisture in sand.
(3) A standard test for indicating the desirability of a fine aggregate.
(4) Effect of dirty aggregates on strength, durability, wear, and surface conditions.
(5) Examination of the distribution of pebbles of siliceous and limestone character in regional deposits of gravel to the end that some workable limits may be placed on requirements for thickness of fireproofing of structural members, and an examination of the necessity of such distinction in thicknesses.

CONCRETE

(1) Method for determining the workability of plain concrete and mortar.
(2) Studies of the various new admixtures for use in concrete.
(3) Water-cement ratio as affected by cements requiring varying amounts of water for normal consistency.
(4) Surface preservation treatment of exposed concrete surfaces.
(5) Long-time tests on effect of admixtures of calcium chloride.
(6) Possibility of producing colloidal or elastic concrete mortar.
(7) Development of satisfactory sand bearing to replace capping of concrete and mortar cylinders.
(8) Effect of clay on watertightness of concrete, the clay being added to the mixing water.
(9) Conditions affecting the disintegration of concrete subjected to freezing temperatures and percolating water.
(10) Development of an accurate, compact, and practical apparatus for making daily field tests to determine the cross-bending strength of concrete used in highway construction.
(11) Flow of concrete under varying conditions of age, loading, humidity, and temperature.
(12) Effect of temperature changes immediately before testing upon concrete strength.
(13) Absorption and expansion of concrete using various coarse and fine aggregates.
(14) Study of causes of scaling from point of view of aggregates, finish, fatigue, vibration, subgrade, and unknown causes.
(15) Efficiency of various forms of apparatus for controlling the uniformity of concrete as manufactured in the field.

REINFORCED CONCRETE

(1) Bond stresses in a bar at various sections along its length, in relation to amount of embedment at the various points considered.
(2) Possibilities in the use of welded connections between reinforcing bars as a means of increasing the strength and economy of reinforced concrete construction.
(3) Coverage of steel necessary against sea air in sea coast structures.
(4) Protection of steel in reinforced concrete against sewer gases.
(5) Vibrations of reinforced concrete members.
(6) Strength of one- and two-way slabs.
(7) Study of the so-called bar joist.

VI. REFERENCES TO PAPERS AND REPORTS ON RESEARCHES PUBLISHED DURING 1928

In compiling the accompanying list of references the aim has been to report only the more noteworthy articles. The references were compiled principally from the following publications:

American Concrete Institute *Proceedings;* American Society Civil Engineers *Proceedings;* American Society Testing Materials *Proceedings;* Ceramic Abstracts; Chemical Abstracts; *Concrete; Engineering News-Record;* Engineering Experimental Station Bulletins; Highway Research Board *Proceedings; Highway Research News; Industrial and Engineering Chemistry; Journal* American Chemical Society; National Bureau of Standards, Washington, D. C.: circulars, scientific papers, technical papers; *Public Roads; Rock Products;* Foreign Technical Publications; *Beton und Eisen,* Berlin; British Institute of Structural Engineers, London; *Canadian Engineer; Concrete and Construction Engineering,* London; *Deutscher Ausschuss fur Eisen-beton,* Berlin; *Der Bauingenieur,* Berlin; Engineering Abstracts of Institution of Civil Engineers, London; Building Science Abstracts, London; *Engineering Journal,* Canada; Forschungsarbeiten auf dem Gebiete des Ingenieurwesens, Berlin; Forscherarbeiten auf dem Gebiete des Eisenbetons, Berlin; Scientific and Industrial Research Technical Papers, London; *Tonindustrie Zeitung,* Berlin; and *Zement,* Berlin.

CEMENT

Cementing Oil Wells:

Cementing Oil Wells to Shut Out Ground Water, by J. F. Hough.
Eng. News-Rec., v. 100, p. 392, Mar. 8, 1928.
Methods and equipment used to combat water intrusion.

Constitution:

Dusting of Melts in the Lime-Alumina-Silica System, by Suzuki and Kasai.
Sci. Papers Inst. Phys. Chem. Res. (Tokyo), v. 7, p. 173, 1927.

Methods of Determining Calcium Carbonate in Cement Mix, by F. Kubik.
Tonind. Ztg., v. 52, p. 678, 1928.
Abstract, *Rock Products*, v. 31, p. 98, Sept. 15, 1928.

Determination of Free Lime, by H. Rathke.
Tonind. Ztg., v. 52, p. 1318, 1928.
Use of tartaric acid in lieu of ammonium acetate to obviate hot titration, and to eliminate the possibility of error due to formation of water during the reaction.

Determination of Free Lime in Slags and Cements, by Diepschlag and Matting.
Zement, v. 17, pp. 1306, 1337, 1373, 1928.
Discussion of the methods of various investigators, including that of Lerch and Bogue.

Effect of Curing on Apparent Free Lime Content of Portland Cement, by A. J. Pool.
Rock Products, v. 31, p. 70, Sept. 15, 1928.
$Ca(OH)_2$ is formed in freshly ground cements by hydrolysis with moisture in air; carbonation takes place at a slower rate.

Determination of Manganese in Lime and Cements and Their Raw Materials, by E. Pujol.
Rev. Mat. Const. et Trav. Pub., v. 23, p. 7, Jan., 1928.

Formation and Solution of Calcium Hydroxide Crystals in Portland Cement, by J. O. Draffin.
Ind. and Eng. Chem., v. 20, p. 311, Mar., 1928.

Hydrated Portland Cement as a Colloid, by A. H. White.
Colloid Symposium Monograph, 1927.
Rock Products, v. 31, p. 50, Apr. 28, 1928.
Study of the nature of the products formed in the hydration process.

Trend of Portland and Accelerated Portland Cement Composition, by E. C. Eckel.
Eng. News-Rec., v. 100, p. 617, Apr. 19, 1928.
Rock Products, v. 31, p. 58, Apr. 28, 1928.
Gap between present composition and pure tri-calcic silicate being closed by manufacturing refinements.

Chemistry of High Early Strength Cements, by H. Kühl.
Tonind. Ztg., v. 52, p. 697, 1928.
Concrete, v. 33, pp. 109, 103, July and Aug., 1928.
Rock Products, v. 31, p. 50, May 26, 1928.

Composition of Cement.
Rock Products, v. 31, p. 52, 1928.
A brief summary, with bibliographical references, is given of various papers on the constitution of cement.

Survey of the Constituents of Portland Cement Clinker, by Güttmann and Gille.
Tonind. Ztg., v. 52, p. 418, 1928.
Tabulated survey.

Review of Literature on Constitution of Portland Cement Clinker, by H. Richarz.
Tonind. Ztg., v. 52, pp. 410, 566, 1928.

Crystals of Technical Portland Cement, by Güttmann and Gille.
Zement, v. 17, p. 296, 1928.

Cement Investigations, by R. Nacken.
Zement, v. 17, No. 1, 1928.
Rock Products, v. 31, p. 50, May 26, 1928.
Paper presented at meeting of German Portland Cement Manufacturers.

Identification of Blast Furnace Slags in Cements, by H. W. Souell.
Zement, v. 17, p. 437, 1928.
Rock Products, v. 31, No. 19, p. 98, 1928.
Hydraulic slags when heated with a solution of lead acetate and acidified with acetic acid, acquire a characteristic brown or black color. Tests of 15 cement clinkers were negative, and this procedure is, therefore, recommended for detecting the addition of slag to cement.

Crystal Structure of Portland Cement, by Janecke.
Zement, v. 17, Jan. 12, 23, 1928.
Pit and Quarry, v. 16, p. 91, April 11, 1928.
Janecke refutes Güttmann and Gille's statement that both Dyckerhoff's dicalcium-silicate and Alite are varieties of mixed crystals. Güttmann and Gille claim they have demonstrated that tricalcium silicate is capable of absorbing more or less alumina, depending upon its condition of formation, and they retain their belief that Alite is a substance composed of mixed crystals.

Composition of Alite, by O. Rebuffat.
Giorn. Chim. Ind. Applicata, v. 9, p. 520, 1927.

Research on Alite, by E. Janecke.
Zement, v. 17, p. 48, 1928.
Rock Products, v. 31, p. 50, May 26, 1928.
Paper presented at meeting of German Portland Cement Manufacturers, 1928.

New Studies on Alite, by H. Kühl.
Zement, v. 17, p. 1303, 1928.

More about Alite, by E. Janecke.
Tonind. Ztg., v. 52, pp. 757, 782, 1928.
Constitution of cement; setting and hardening processes.

Combination of Lime in Portland Cement Compounds, by Hansen and Bogue.
Ind. and Eng. Chem., v. 19, p. 1260, 1927.
Rock Products, v. 46, Mar. 3, 1928.
Reprinted as Paper No. 10, P.C.A. Fellowship, Bureau of Standards.

Hydrolysis of Compounds Which May Occur in Portland Cement, by Lerch and Bogue.
Jl. of Phys. Chem., v. 31, p. 1627, 1927.
Reprinted as Paper No. 11, P.C.A. Fellowship, Bureau of Standards.

Further Studies of Portland Cement Compounds by the X-Ray Diffraction Method, by W. C. Hansen.
Jl. Am. Ceramic Soc., v. 11, p. 68, 1928.
Reprinted as Paper No. 12, P.C.A. Fellowship, Bureau of Standards.

Studies on the System Calcium-Oxide-Alumina-Ferric Oxide, by Hansen, Brownmiller, and Bogue.
Jl. Am. Chem. Soc., v. 50, p. 396, 1928.
Reprinted as Paper No. 13, P.C.A. Fellowship, Bureau of Standards.

Equilibrium Studies on Alumina and Ferric Oxide and Combinations of These with Magnesia and Calcium Oxide, by Hansen and Brownmiller.
Am. Jl. of Science, v. 15, p. 225, 1928.
Reprinted as Paper No. 14, P.C.A. Fellowship, Bureau of Standards.

Phase Equilibria in the System $2CaO \cdot SiO_2 - MgO - 5CaO \cdot 3Al_2O_3$, by W. C. Hansen.
Jl. Am. Chem. Soc., v. 50, p. 3081, 1928.
Reprinted as Paper No. 18, P.C.A. Fellowship, Bureau of Standards.

Fineness:

Air Analyzer for Determining Fineness of Cement.
Rock Products, v. 31, p. 70, June 9, 1928.
Its use in the laboratory of the Lehigh Portland Cement Company.

Measurement of Finest Particles in Portland Cement with the Wiegner Sedimentation Apparatus, by Kühl and Tokune.
Zement, v. 17, p. 256, 1928.

Fineness Testing of Portland Cement.
Quarry, v. 33, p. 148, 1928.
Use of an elutriator.

Grain Size of Cement, by Werner and Giertz-Hedstrom.
Zement, July, 1928.
Pit and Quarry, v. 16, p. 81, Aug. 1, 1928.
Principle of falling of particles through a liquid, in this case alcohol.

Method of Measuring Particle Sizes in Ground Powders, by J. V. Ramsden.
Jl. Oil and Color Chem. Assn., v. 11, p. 16, 1928.

Measurement of Particle Size by X-Rays, by A. L. Patterson.
Zeits. Krystallogr. Mineral, v. 66, p. 637, 1928.
Zeut., v. 99, p. 2692, 1928.
Based on measurement of width of lines of diffraction patterns.

Fineness of Cement (Report of Sub-Comm. III of A.S.T.M. Committee C-1).
Proc. Am. Soc. Testing Mat., v. 28, 1928.
Data of fineness determinations on 32 cements submitted by 47 cooperating laboratories.

Graphical Analysis of Fineness Distribution Curves for Pulverized Materials, by L. T. Work.
Reprint, Thesis submitted for Ph.D. Degree, Columbia University, 1928.

High-Early Strength Portland Cement:

Manufacture of High-Early Strength Portland Cement, by G. P. Dieckmann.
Rock Products, v. 31, p. 58, May 26, 1928.
With particular reference to Novo cement, manufactured in Germany.

High-Early Strength Portland Cements in Europe, by E. C. Blanc.
Concrete (CMS), v. 32, p. 101, Feb., 1928.
Discussion of super portland cements, the manufacturing cost of which is quoted as slightly higher than that of ordinary portland cement, with the advantage of rapid hardening.

Advantages of Quick-Hardening Cement, by H. S. Mattimore.
Concrete, v. 32, p. 45, June, 1928.
Discussion of several methods of securing high early strength concrete for pavement construction.

Gain in Strength of High Strength Portland Cements Cured in Air and in Water, by Haegermann.
Tonind. Ztg., p. 119, 1928.
Concrete (CMS), v. 32, p. 106, Mar., 1928.
Differences in character, manufacture, and behavior between high early strength and ordinary portland cements.

Early Strength Concrete Made with Ordinary Portland Cement, by W. Cahill.
Eng. News-Rec., v. 100, p. 410, 1928.
Strict observance of water-cement ratio and use of rich mix.

Modern Cements, by P. H. Bates.
Eng. News-Rec., v. 100, p. 887, June 7, 1928; p. 932, June 14, 1928.
White cement, waterproofed, and masonry cements; high early strength and portland cements.

High Early Strength Developed by New Portland Cement.
Eng. News-Rec., v. 101, p. 226, 1928.
A new portland cement has been produced which is said to develop a high strength in 24 hours. Precast concrete piles have been driven seven days after casting without showing signs of cracking.

Special Rapid Hardening Cements, by E. Rengade.
Rev. Mat. Const. et Trav. Pub., No. 225, p. 203, 1928.
High early strength cement mortars give strengths at 2 and 7 days equal to the 7 and 28-day strengths of ordinary portland cement mortars.

Manufacture:

Preparation of Cement Raw Mix, by M. Elber.
Rev. Mat. Const. et Trav. Pub., v. 23, p. 1, Jan., 1928.
Having uniform raw materials, clay and limestone, and using wet process, control of mix requires but 1 hr. of chemist's time per day.

Iron Oxide *vs.* Alumina as a Fluxing Agent in the Manufacture of Portland Cement, by A. J. Blank.
Rock Products, v. 31, p. 48, May 26, 1928.
Alumina content in raw materials of no marked benefit in this particular case, as a fluxing agent in the kilns, while the opposite is true of a semi-high iron oxide content.

Changes Occurring in a Calcareous-Argillaceous Mix during the First Phase in the Burning of Portland Cement, by E. Monath.
Rev. Mat. Const. et Trav. Pub., v. 23, p. 81, Mar., 1928.

Dry Blending of Raw Materials.
Concrete, v. 32, p. 105, June, 1928.

Portland Cement Manufacture, by A. C. Davis.
Cement and Cement Manuf., v. 1, p. 39, Oct. 1928.
Early history.

Progress in Making Lime and Cement and Their Effect on Masonry, by J. Bolomey.
Le Ciment, v. 33, p. 323, 1928.

Gypsum and Anhydrite in the Manufacture of Portland Cement, by R. K. Meade.
Rock Products, v. 31, p. 55, Nov. 24, 1928.
Effect of various forms of calcium sulfate as retarders; relative importance of anhydrite and gypsum as retarders.

Manufacture of Cement at the Factory of the Brazilian Portland Cement Company.
Bol. Soc. Chim. Sao Paulo, v. 1, p. 7, 1928.
General discussion of manufacturing methods applied to raw materials there available.

New Developments in Cement Manufacture, by E. Shaw.
Rock Products, v. 31, p. 79, May 12, 1928.
Some minor problems with which the industry is confronted.

Electrical Unwatering of Portland Cement Slurries, by L. G. Hall.
Cement Mill and Quarry, v. 32, p. 54, May, 1928.
Fundamental factors of process.

Recovery of Heat Loss in the Manufacture of Cement, by G. Smith.
Power, Jan. 31, 1928.
Le Ciment, v. 33, p. 207, May, 1928.

Research—General:

Portland Cement of Today and Tomorrow, by A. C. Davis.
Cement and Cement Manuf., v. 1, p. 3, Sept., 1928.

Report on Recent Research, by Nacken.
Zement, v. 17, p. 1231, Aug. 16, 1928.
Summary of German and American experimental studies of mineral and chemical composition of portland cement clinker; crystallography of cement.

Improvement in Cement and Concrete, by H. Richarz.
Zement, v. 17, pp. 1348, 1377, 1411, 1928.
Résumé of efforts to improve quality of concrete during the past fifty years by means of admixtures, new equipment, longer mixing period, tamping, and reduction of amount of mixing water.

Portland Cements.
Pamphlet, Zurich, 1928.
Results of tests at Federal Laboratory for Testing Materials made during 1923–1927.

Setting and Hardening:

Electrical Method for Measuring the Setting Time of Cement, by Yosomatsu Shimizu.
Science Report No. 1, Tohuku Imperial Univ., Series 1, v. 17, Jan., 1928.
Change in electrical resistance during setting and hardening of portland cement measured to determine the setting time; effect of temperature on rate of setting.

Manufacturing Conditions Influencing Setting Time of Portland Cement, by R. K. Meade.
Rock Products, v. 31, p. 58, Mar. 31, 1928.
Effecting of seasoning, heating, fineness of cement and retarder, and influence of alkalis and lime ratio on setting time.

Hardening of Mortars and Concrete, by Bolomey.
Bull. Tech. de la Suisse Romande, Nov. 5, 1927.
Ciment, v. 33, p. 117, Mar., 1928.
Effect of quality of cement and duration of hardening.

Digest of Literature on Nature of Setting and Hardening Processes in Portland Cement, by R. H. Bogue.
Rock Products, v. 31, p. 69, May 12; p. 62, June 9; p. 61, July 7; p. 67, Aug. 4; p. 67, Sept. 1; p. 65, Sept. 29; Paper No. 17, P.C.A. Fellowship, 1928.
Historical introduction and early views; theory of crystallization from supersaturated solutions; chemical significance of rate of setting.

Temperature Changes during the Setting of Alumina and Portland Cements, by Haller.
Betonstrasse, p. 130, May, 1928.

Apparatus for Determining the Time of Setting of Cements, by L. Liautaud.
Arts et Metiers, No. 91, p. 146, April, 1928.
Rev. Mat. Const. et Trav. Pub., No. 226, p. 256, July, 1928.

Comparative Setting Time Observations with the Vicat Needle and the "Automat," by Nitzsche.
Tonind. Ztg., v. 52, p. 1036, 1928.

Alternate Hardening of Portland Cement, by H. Kiehl.
Pit and Quarry, v. 16, p. 43, No. 9, 1928.
Time at which minimum strength of cement samples was attained when hardened in air and under water, and the influence of alternate hardening process.

Effect of Retarders on Portland Cement Clinkers, by E. E. Berger.
Rock Products, v. 31, p. 50, Dec. 8, 1928.
Plan of investigation; experiments; results; interpretation; conclusions.

Studies on the Processes Occurring during Setting and Hardening of Hydraulic Cements, by K. Biehl.
Zement, v. 17, pp. 487, 824, 1928.

Effect of Litharge on Hardening of Portland Cement, by B. Garre.
Zeits. anorg. allgem. Chem., v. 169, p. 305, 1928.

Failure of Lean Concretes to Harden, by B. Garre.
Zeits. anorg. allgem. Chem., v. 169, p. 301, 1928.
Quantities of salts in aggregate or mixing water which would not noticeably affect rich concrete may prevent hardening in lean concrete.

Action of Salts on Cement Setting, by Alistar and Roussel.
Annales Off. Nat. Combustibles Liquides, v. 3, pp. 741, 754, July and Aug., 1928.
Action of alkaline chlorides, sulfates, and carbonates.

Soundness:

Cracking of Glass Plates Caused by Normal Cement Pats, by F. Schott.
Zement, v. 17, pp. 1145, 1169, 1928.
Cracking occurred not because of contraction of the paste nor the difference in coefficient of expansion of the cement and glass, but because the pat adhered to the glass plate, except when the tensile strength of the glass was very high. Unsound cements did not usually adhere to the plate, and for this reason it is suggested that a test for soundness be developed on this basis.

Specifications:

International Standards for Cement, by C. R. Platzmann.
Zement, v. 17, 1928.
Rock Products, v. 31, p. 97, Jan. 21, 1928.
Concrete (CMS), v. 32, p. 103, Feb., 1928.
Critical review of standards of 25 countries with a view to international standardization.

New Belgian Standards for Cement.
Rock Products, v. 31, p. 83, Jan. 7, 1928.
Belgian specifications revised and put into effect several months before the revised German specifications. Standards include ordinary portland cement and quick-hardening portland cement.

New Polish Cement Standards.
Zement, v. 17, p. 180, 1928.

Japanese Specifications for Portland Cement.
Zement, v. 17, p. 224, 1928.

Italian Specifications for Cement, Concrete, and Reinforced Concrete Construction.
Zement, v. 17, p. 337, Mar. 1, 1928.
Le Strade, Jan., 1928.
Rev. Mat. Const. et Trav. Pub., v. 23, p, 151, April, 1928.

European High Strength Cement Specifications and Facts, by E. C. Eckel.
Cement, Mill, and Quarry, v. 32, p. 89, May, 1928.
Results of tests made on high strength cements produced in European plants; inadequacy of present specifications.

Concrete and Reinforced Concrete, by T. H. Bryce.
Ferro-Concrete, v. 20, p. 67, Sept., 1928.
Specifications for cement used in England.

Specifications for Delivery and Testing Cement.
Tekniska Forening. i Findland Forh., v. 48, p. 152, July, 1928.
Cement standards proposed by cement committee of Finland.

New Italian Cement Specifications, by H. Dewidels.
Tonind. Ztg., v. 52, p. 1348, 1928.
The earlier specification recognized two qualities of cement, but only one is accepted now, for which the minimum compressive strength has been raised from 250 to 280 kg. per sq. cm. at 28 days.

New Dutch Standard Specifications for Portland, Iron Portland, and Blast Furnace Cements.
Zement, v. 17, p. 1105, 1928.

Tests and Test Methods:

Modern Cement and Cement Testing, by D. B. Butler.
Cement and Cement Manuf., v. 1, p. 9, Sept., 1928.

Testing of Portland Cement, by R. H. H. Stanger.
Cement and Cement Manuf., v. 1, p. 13, Sept., 1928.

Testing Cements with Plastic Mortars, by A. Kleinlogel.
Zement, v. 17, p. 102, Jan. 19, 1928.

Cement Tests Made by Thirty-Two Laboratories.
Pamphlet, Japan Soc. Portland Cement Mfrs., 3 pages, 1927.

Tests for Tensile and Compressive Strengths and Quality of Cement, by A. Dahlgren.
Rev. Mat. Const. et Trav. Pub., No. 220, p. 22, 1928.
It is pointed out that in the new German specifications, minimum 7 and 28-day compressive strengths are specified as well as minimum 7-day tensile strength.

Plastic Mortar Compression Test for Cement, by E. M. Brickett.
Proc. Am. Soc. Testing Mat., v. 28, 1928.
Suggested specification test for portland cement which will give true indication of concrete-making qualities. Two mortar cubes are made using one part cement to 2.75 parts sand by weight, gaged to the same water-cement ratio as the concrete to which the strength is to be compared.

Accelerated Cement Testing.
Pit and Quarry, v. 16, p. 85, May 23, 1928.
Steam curing to obtain 28-day strength in 2 days.

Adjustable Dash Pot for Cement Testing.
Rock Products, v. 31, p. 61, Jan. 7, 1928.
Can be attached to the usual form of Vicat needle apparatus to control mechanically the lowering of the needle rod to the cement paste, thereby assuring a standard condition of testing.

Cement Testing in Italy.
Tonind. Ztg., v. 52, No. 56, p. 1138, 1928.
Criticism of present methods of testing, and proposed new methods.

Tentative Standard Methods of Sampling and Testing Highway Materials.
Bull. 1216, U. S. Dept. Agriculture, Sept., 1928.

Effect of Quality of Portland Cement on the Strength of Concrete, by F. H. Jackson.
Concrete, v. 33, p. 43, Nov., 1928.
Strength of concrete varies with character of cement used.

Recent Improvements in the Strength and Constructive Value of Portland Cement, by D. B. Butler.
Structural Eng., v. 6, p. 333, Nov., 1928.

Wear of Cement Particles in Sieve Tests, by Foerderreuther and Haegermann.
Tonind. Ztg., v. 52, p. 1766, Nov. 3, 1928.
Elaborate experimental study of abrading effect of sifting on cement particles; formation of rules for cement sieve tests, particularly as to duration of sifting.

Unit Weight:

Unit Weight of Portland Cement, by Haegermann.
Zement, v. 17, p. 379, Mar. 8, 1928.

Cements Other than Portland:

Development of Alumina Cement Industry in Europe, by E. Blanc.
Concrete (CMS), v. 32, p. 113, Apr., 1928.

Ciment Fondu Concrete.
Engineer, v. 145, p. 352, Mar. 30, 1928.

New Investigations of Aluminous Cements, by Feret.
Rev. Mat. Const. et Trav. Pub., v. 23, pp. 25 and 135, 1928.

Investigations of Inaptitude of Certain Aluminous Cement Concretes, by R. Feret.
Le Genie Civil, v. 92, p. 210, Mar. 3, 1928.

Present State of Our Knowledge of Aluminous Cements, by C. Blanchet.
Rev. Mat. Const. et Trav. Pub., No. 220, p. 8, Jan., 1928.

Producing High Alumina Slags for Alumina Cement, by T. L. Joseph.
U. S. Bureau of Mines Reports, Serial No. 2869, 1928.
Rock Products, v. 31, p. 64, June 9, 1928.

Conduits for Underground Telephone Lines, by Rengade.
Le Ciment, v. 33, p. 15, Jan., 1928.
Concrete and Const. Eng., v. 23, p. 228, Mar., 1928.

Recent Observations of Defects in Concrete Made from Alumina Cement. by Coyne and Freysinnet.
Le Genie Civil, v. 93, p. 140, Aug. 11, 1928.
Cites experiences of Corde Bridge in Finistarre.

Failure of Alumina Cement, by J. Silberstein.
Nat. Eng., v. 32, p. 270, June, 1928.
Bridge near Brest, France, failed due to alumina cement concrete in which sea water had been used for mixing.

Action of Large Excess of Water on Aluminous Cement, by H. Vierheller.
Tonind. Ztg., v. 52, p. 611, 1928.

Aluminous Cement in Practice, by F. Buchi.
Beton und Eisen, v. 27, p. 174, 1928.
Flexural test with beams made on job should give reliable indications of the strength of concrete made with aluminous cement.

Metallurgical Slags and Cements.
Rev. Mat. Const. et Trav. Pub., p. 250, 1928.
At Atrebach in the Sarre, slag is made into portland cement having the composition: CaO 67.3, SiO_2 18.9, Al_2O_3 6.6, Fe_2O_3 2.2, and MgO 2.6%. The clinker is then ground with granulated slag to form metallurgical cements which meet the Parisian specifications for strength. The method of manufacture is described.

Slag Cement, Its Properties and Applications, by J. C. de Langavant.
Rev. Mat. Const. et Trav. Pub., No. 222, 226, pp. 87, 250, Mar. and July, 1928.

European Mixed Portland Cements, by E. C. Blanc.
Concrete (CMS), v. 32, p. 104, Mar., 1928.
Iron portland cements; sand cements; data on their performance; comparisons.

Alteration in the Definition of Blast-Furnace Cement.
Baunormung, v. 7, p. 16, 1928 (Supp. to *Bauing.*, v. 9, No. 17, 1928).
Now defined as a hydraulic binding material consisting of basic blast-furnace slag granulated by rapid cooling, and 15–69% portland cement by weight.

Manufacture of Super Cement, by N. C. Kyriacou.
Rev. Mat. Const. et Trav. Pub., No. 230, p. 417, Nov., 1928.
Manufacture in rotary kilns.

Super Cement.
Rev. Mat. Const. et Trav. Pub., No. 230, p. 414, Nov., 1928.
Translation into French of circular from Super Cement Co., Toronto.

Super Cements and Cold Water, by H. Vierheller.
Zement, v. 17, p. 892, 1928.
Time of set and early strength of alumina cements are only slightly affected by curing water close to the freezing point, while this treatment of super cement is detrimental. The super cement reaches its normal strength at 28 days when stored in ice water, whereas the alumina cement has not reached its normal strength.

Natural Cement of Vassy, by G. B. de l'Isle.
Rock Products, v. 31, p. 63, Sept. 29, 1928.
Natural cement rock of Vassy, France, burned in shaft kilns shows remarkable properties that could well be imitated in portland cements; chemical and physical properties.

Portland Jurament, by H. Klebs.
Tonind. Ztg., v. 52, p. 1384, 1928.
A cement consisting of portland cement clinker, blast-furnace slag, and oil shale residue. It is claimed that this cement develops strengths equal to or greater than normal portland cements, and that it is more resistant to the action of sulfates, acid waters, and magnesium chloride solutions.

Plain Concrete

Admixtures:

Effect of Sugar upon Tensile Strength of Portland Cement Mortar, by Morrison and Boyd.
Concrete, v. 32, p. 26, Jan., 1928.
Tensile strength of 1–3 mortar was progressively decreased by addition of sugar up to about 0.3 to 0.4% of the weight of sand.

Tests of the Influence of Trass and Other Powdered Minerals on the Tensile and Compressive Strength of Cement Mortars, Their Permeability, and Resistance to Chemical Attack, by O. Graf.
Zement, v. 17, Mar. 15, 1928; p. 492, Mar. 22, 1928; p. 543, Mar. 29, 1928.

Behavior of Trass Cement in Contact with Harmful Solutions, by H. Bach.
Tonind. Ztg., v. 52, p. 1058, 1928.
Trass cement concretes are less likely to be attacked by harmful solutions than are other concretes.

Lime in Cement-Trass Mixtures, by T. Klehe.
Tonind. Ztg., v. 52, p. 1037, 1928.
A mixture of 1 part by weight of portland cement, 0.5 of trass, 0.75 of lime paste, and 5 parts of standard sand developed strength equal to that of a good cement-sand mix.

Danger of Gypsum in Concrete Construction, by Fammler.
Rev. Chambre Syndicate, Apr., 1928.
Ciment, v. 33, p. 203, May, 1928.

Coloring of Portland Cement, by A. P. Lawrie.
Jl. Royal Inst. British Arch., v. 35, p. 369, 1928.

Colored Portland Cements, by G. F. Palmer.
Building, v. 3, p. 187, 1928.

Color of Cement, by M. Elber.
Rev. Mat. Const. et Trav. Pub., No. 221, p. 41, 1928.

Colored Concrete.
Practical Builder, v. 3, No. 8, p. 315, 1928.
Selection of aggregate with a view to color required, and use of lowest possible quantity of mixing water are most important factors. Bricks having a weathered appearance may be made of colored concrete; for this purpose old bricks form a good aggregate.

Clay Concrete, by F. Schrader.
Gross Berliner Bauzeitung, No. 49, p. 6, 1927.
Zement, v. 17, p. 13, 1928.
Substitution of clay for part of fine aggregate to increase frost resistance and reduce shrinkage cracking.

Diatomite.
Bull. 691, Canada Dept. of Mines, 1928.
Occurrence, preparation, and uses, including its use as an admixture in concrete, giving waterproofing tests and experiments of the Department of Public Works.

Shrinkage Effect of Celite in Mortar and Concrete, by A. S. Levens.
Eng. News-Rec., v. 101, p. 507, Oct. 4, 1928.

Admixtures for Concrete and Mortar, by H. E. Schubert.
Bauing., v. 9, p. 324, 1928.
Composition and method of use of 11 German integral waterproofers for concrete and mortar.

Effect of Admixtures on the Water-Cement Ratio Strength Relation of Concrete, by G. Conahey.
Proc. Am. Soc. Testing Mat., v. 28, 1928.

Age:

Equation for Predicting Strength of Concrete, by F. N. Wray.
Eng. News-Rec., v. 101, p. 291, Aug. 23, 1928.
Age-strength formula.

Aggregates:

Long-Time Tests to Compare Various Coarse Aggregates, by P. J. Freeman.
Eng. News-Rec., v. 99, p. 879, 1927.
Rock Products, v. 31, p. 75, Mar. 3, 1928.
Comparison of slag, limestone, granite, trap rock, and gravel. Tests made at 14, 30, 60, and 180 days, 1, 2, 3, 4, 5, and 10 years.

Comparative Tests of Crushed Stone and Gravel Concrete in New Jersey, by F. H. Jackson.
Rock Products, v. 31, p. 95, Mar. 17, 1928.
Natl. Sand and Gravel Bull., v. 9, p. 13, Mar., 1928.
Crushed Stone Journal, Mar. 1928.
Discussion by Goldbeck, *Rock Products*, v. 31, p. 91, Apr. 14, 1928.
Discussion by Stanton Walker, *Rock Products*, v. 31, p. 73, Mar. 31, 1928.
Discussion by Stanton Walker and A. T. Goldbeck, *Concrete*, v. 32, p. 23, 1928.

Comparative Tests of Gravel and Crushed Stone for Concrete Making, by R. V. Frost.
Statens Provning. (Stockholm), No. 40, 1928.
Tests by Swedish Concrete Institute lead to conclusion that gravel and crushed stone are practically of equal value as concrete aggregates with regard to strength and deformation under load.

Effect of Moisture on Toughness of Rock.
Public Roads, v. 9, p. 183, Nov., 1928.
Materials tested were loose-textured, granular sandstone; soft, crystalline limestone; soft, amorphous limestone; hard, siliceous limestone; trap; and granite. Toughness of the dry rock was the same as when the rock was thoroughly saturated with water.

Relation between Absorption and Soundness Tests of Sedimentary Rock, by D. O. Woolfe, Jr.
Public Roads, v. 8, p. 225, Dec., 1927.
Rock Products, v. 31, p. 39, Jan. 7, 1928.
150 rocks tested for soundness and absorption; of samples tested 43% were unsound; of samples with absorption greater than 2%, 82% were sound.

Rate of Absorption of Limestone, by H. F. Kriege.
Rock Products, v. 31, No. 9, p. 62, 1928.
Brit. Lime, v. 2, No. 8, p. 221, 1928.
Precise determination of absorption capacity of stone used as coarse aggregate necessary in proportioning concrete of a given consistency.

Influence of Durability Tests on Strength of Concrete Aggregate.
Tech. News Bull. No. 134, Nat. Bureau of Standards, p. 87, 1928.
Tests on cylinders about 2 in. in diameter and 2 in. high and on 2-in. cubes cut from limestone blocks selected at random. The samples were boiled in water for 6 hours and then dried at 110 deg. C. for 17 hours, this cycle being repeated 100 times. Comparative compressive strength tests carried out with untreated samples cut from the same blocks showed an average reduction of 38% in the strength as a result of the cycles of boiling and drying.

Relation between Standard Abrasion Tests for Stone and Gravel, by D. O. Woolf.
Public Roads, Sept., 1928.
Crushed Stone Jl., v. 4, p. 3, Oct., 1928.
Abrasion tests on rock and synthetic gravel yield unexpected results; abrasion tests were made by three methods; results indicate desirability of modifying or supplanting abrasion test for gravel.

Usefulness of Petrology in Selection of Limestone, by G. F. Loughlin.
Rock Products, v. 31, p. 50, Mar. 17, 1928.
Quarry failures and poor concrete could have been avoided if the mineral composition and texture of stone had been determined and appreciated.

Concrete Strength from Aggregates of Unusual Grading, by P. H. Sherlock.
Concrete, v. 32, p. 43, Mar., 1928.
Test procedure; test results; conclusions.

Effect of Grading of Gravel on Voids.
National Sand and Gravel Bull., v. 9, p. 27, Oct., 1928.
Combination of three sizes of gravel giving minimum voids.

Relation of Grading and Voids in Sand.
Rock Products, v. 31, p. 56, June 9, 1928.
Use of tri-axial diagrams for sand analysis.

Effect of Shape and Character of Coarse Aggregate on Strength of Concrete, by F. C. Lang.
Concrete, v. 32, p. 37, Mar., 1928.
Account of tests made by Minnesota Highway Department.

Effect of Surface Condition of Aggregate, by Bolomey.
Bull. Tech. Suisse Romande, 1928.
Rock Products, v. 31, No. 11, p. 89, 1928.

Methods of Making Deval Abrasion Tests of Aggregates, by S. Walker.
Proc. Am. Soc. Testing Mat., v. 28, 1928.
Effects of such variable factors as (1) methods of measurement of wear. (2) number of cylinder revolutions, (3) weight of sample, (4) number of balls in abrasive charge, (5) grading of sample, (6) shape of particles, and (7) type of aggregate.

Report of Section of Committee C-9 on Abrasion Tests of Concrete Aggregates.
Proc. Am. Soc. Testing Mat., v. 28, 1928.
Various modifications of the Deval test are discussed, and a description given of investigations carried out with a view to developing a modification suitable for testing gravel.

Concrete and Concrete Aggregates (Report of Comm. C-9).
Proc. Am. Soc. Testing Mat., v. 28, 1928.
Proposed method for testing concrete materials, including soundness, abrasion, colorimetric, moisture, and field tests.

Proposed Tentative Specifications for Concrete Aggregates (Report Comm. C-9).
Proc. Am. Soc. Testing Mat., v. 28, 1928.
Proposed maximum limits for deleterious substances in fine and coarse aggregates, and grading of sand.

Limestone Screenings in Concrete, by Warford, Wil, and Peyton.
Concrete and Const. Eng., v. 23, p. 29, Jan., 1928.

Use of Florida Aggregates in Making Concrete, by H. R. Albion.
Florida Eng. and Contr., v. 5, p. 50, Apr. 1928.

Louisiana Gravels as Coarse Aggregates for Concrete Pavements and Pavement Foundations, by J. H. Bateman.
Bull. 3, Louisiana State Univ., May, 1928.
With the same sand and fixed mortar proportions, the amount of coarse aggregate which can be used without causing under-sanded concrete varies directly according to the size of the coarse aggregate. Small gravel showed slightly more wear resistance than moderately coarse gravel, and concrete with small gravel is more plastic and workable than concrete with large. The use of ¾-in. Louisiana gravel is not justified economically.

Properties of Breeze and Clinker Aggregates and Methods of Testing Their Soundness.
Bull. No. 5, Dept. Sci. and Industrial Research, 1928.
Methods of testing which are designed to be applicable without laboratory apparatus to simplify field testing.

Slag, Coke Breeze, and Clinker as Aggregates, by Lea and Brady.
Dept. Sci. Ind. Research (Brit.), No. 10, 1927.

Symposiums of the National Slag Association, 1928:
Symposium 1—Absorption in Slag and Slag Concrete.
Symposium 2—Resistance of Blast Furnace Slag to Abrasion and Wear.
Symposium 3—Action of Slag under High Temperatures and Fire.
Symposium 4—Manufacture and Use of Slag Products.
Symposium 5—Bond in Slag and Slag Concrete.
Symposium 6—Preparation of Slag for the Market.
Symposium 7—Use of Slag in Concrete Roads and Pavements.
Symposium 8—Use of Slag in Bituminous Construction.
Symposium 9—Use of Slag Ballast for Railways.
Symposium 10—Is There Any Corrosive Quality in Slag?
Symposium 11—Chemical and Petrographic Composition of Slag.

Regeneration Process for Making Concrete from Blast-Furnace Slag, by R. Schonhofer.
Beton und Eisen, v. 27, p. 129, 1928.

Coal Residues in Concrete (Editorial).
Ferro-Concrete, v. 20, p. 98, Nov., 1928.
A joint committee representing the building industries of Great Britain has reached the conclusion that coal residues are in general unsatisfactory materials for use as aggregates, there being three primary causes of defects: (1) concretes made with such residues are liable to expansion on setting, or later expand after absorbing moisture from the atmosphere, or otherwise (2) the high permeability of concrete made with coal residues permits the access of air and moisture to embedded steel, with consequent risk of corrosion, and (3) the presence of sulfur compounds may lead to corrosion under conditions favorable to chemical action.

Effect of Time and Condition of Curing on the Strength of Asbestos-Cement Shingles, by O. Kallauner.
Zement, v. 17, p. 99, No. 3, Jan. 19, 1928.

Asbestos-Cement Service Pipes, by C. Campbell.
Master Builder, No. 792, p. 19, 1928.
Resistance to corrosion; absence of contamination of water; impermeability to gas; ease in handling.

Sawdust Concrete.
Concrete and Const. Eng., v. 23, p. 642, Oct., 1928.
Concrete was reinforced with steel mesh and round bars. Total weight of a pair of gates including the hinges and bolts was 2343 lb. A month after casting the weight was reduced to 1944 lb. showing that half the quantity of water used had been lost by evaporation.

Sawdust and Cork Tested as Concrete Materials by British Research Board.
Concrete, v. 32, p. 39, June, 1928.

Sawdust as Concrete Aggregate.
Concrete, v. 33, p. 18, Aug., 1928.
European uses; mineralized sawdust; strength and fire tests; nailability; volume changes.

Sawdust as Concrete Aggregate.
Contract Record, v. 42, p. 850, Aug. 15, 1928.
Mineralized sawdust is used to counteract shrinkage and render material incombustible; Columbia University tests.

Bulking of Sand with Varying Moisture Content, by E. Standt.
Zement, v. 17, p. 1077, 1928.
Bulking of 2 sands of similar screen analysis was from 25 to 38% when moisture content was 4 to 5% in each case. Higher moisture content decreased bulking effect.

New Moisture Meter for Aggregates.
Cement Mill and Quarry, v. 32, p. 33, Sept., 1928.
Compact device used to measure water in aggregate and to determine the fineness modulus.

Determination of Specific Gravity, Surface Moisture, and Voids in Fine Aggregate for Use in Concrete Mixtures, by C. M. Chapman.
Concrete Products, v. 34, p. 53, Apr., 1928.
Use of the Chapman flask, a double-bulb glass flask having a graduated neck and a calibrated volume mark on the constriction between the upper and lower bulbs.

Simple Device for Determining Specific Gravity and Moisture in Aggregates.
Rock Products, v. 31, p. 57, Mar. 3, 1928.
A pycnometer or specific gravity flask consisting of an ordinary mason jar with a metal conical top.

Bond:

Bond between Concrete and Hollow Tile, by J. C. Oleinik.
Eng. and Contr., v. 67, p. 19, Jan., 1928.
Results of tests on effect of (1) absorption of tile, (2) moisture content of tile at time of placing concrete, and (3) proportions, consistency, and curing conditions of concrete on bond strength of joints between concrete and hollow tile.

Experimental Tests of Concrete-Steel Bond, by Edwards and Greenleaf.
Proc. Am. Soc. Testing Mat., v. 28, 1928.
Results show wide range of bond strength, probably indicating effect of "mother rock" origin of sands and of their granulometric composition; general effect of use of excessive water content in concrete mix; desirability of more complete investigation of factors affecting concrete-steel bond and its reliability under conditions involving variations in application of loads and impact.

Brick, Block, and Tile:

Specifications for Concrete Stone, by C. Van de Bogart.
Proc. Am. Concrete Inst., 1928.
Concrete, v. 32, p. 34, Mar., 1928.

Drying Concrete Brick to Take Out the Shrinkage, by L. E. Grube.
Proc. Am. Concrete Inst., v. 24, p. 451, 1928.

Chemical Analysis:

Determination of the Quantity of Cement in a Sample of Mortar or Concrete.
Rev. Mat. Const. et Trav. Pub., v. 23, p. 147, Apr., 1928.
Determination of soluble silica and its application to estimation of the cement content of mortar or concrete.

Estimation of Amount of Cement Present in Hardened Mortar or Concrete, by D. Meneghini.
Annali della R. Scuola d'Ingegneria de Padova, Nov., 1927.
Le Ciment, v. 32, p. 461, Dec., 1927.

Procedure for the Analysis of Cinder Concrete, by J. L. Heitzman.
Concrete, v. 32, p. 46, June, 1928.

Determination of Proportions of Constituents in Concrete, by L. G. Garmick.
Public Roads, v. 9, p. 88, June, 1928.
Roads and Streets, v. 68, p. 339, July, 1928.

Consistency and Workability:

Cement as a Factor in the Workability of Concrete, by Bates and Dwyer.
Proc. Am. Concrete Inst., v. 24, p. 43, 1928.

Methods of Measuring Workability of Concrete, by Smith and Conahey.
Proc. Am. Concrete Inst., v. 24, p. 24, 1928.

Measurement of Workability of Concrete, by G. A. Smith.
Proc. Am. Soc. Testing Mat., v. 28, 1928.
Penetration apparatus and results. Device consists essentially of a mold for holding a batch of freshly mixed concrete into which three ½-in. molds are driven by means of a hammer falling a constant distance. The workability indices are based on the average number of blows required to give an 11-in. penetration.

Determination of Workability of Concrete, by Purrington and Loring.
Proc. Am. Soc. Testing Mat., v. 28, 1928.
Determination of workability of a concrete mixture by measuring the power consumed in mixing the concrete. A concrete mixer is driven by a suitable motor and the load applied to the mixer measured by a polyphase wattmeter supplied with proper transformers.

Workability and Durability of Concrete, by R. W. Atwater.
Proc. Am. Concrete Inst., v. 24, 1928.
Eng. and Contr., v. 67, p. 131, Mar., 1928.
Canadian Eng., v. 55, p. 279, Sept. 4, 1928.

What Workability Means to the Contractor, by N. L. Doe.
Eng. and Contr., v. 67, p. 128, Mar., 1928.
Concrete, v. 32, p. 29, Apr., 1928.

Water as a Factor in Workability, by R. L. Bertin.
Eng. and Contr., v. 67, p. 133, Mar., 1928.

Workability of Portland Cement Pastes.
Tech. News Bull., Nat. Bureau of Standards, No. 132, p. 53, Apr., 1928.

New Method for Determining Plasticity of Mortar, by C. Biffi.
Le Strade, p. 75, Mar., 1928.

Gradation and Character of Aggregates as a Workability Factor, by A. T. Goldbeck.
Eng. and Contr., v. 67, p. 270, May, 1928.
Outline of grading possibilities given in workability symposium.

Crazing:

Studies on Crazing of Portland Cement Mortars, by Bates and Jumper.
Proc. Am. Concrete Inst., v. 24, p. 179, 1928.

Crazing in Concrete and Growth of Hair Cracks into Structural Cracks, by White, Aagaard, and Christensen.
Proc. Am. Concrete Inst., v. 24, p. 190, 1928.

Cracks in Concrete Surfaces, by E. P. Parry.
Building, v. 3, p. 137, 1928.
Hair cracks attributed to segregating effect of troweling which leads to the formation of a thin surface film of practically neat cement.

Curing:

Comprehensive Concrete Paving Curing Tests Now in Progress in Tennessee.
Public Roads, v. 9, p. 184, Nov., 1928.
Eng. News-Rec., v. 101, p. 812, Nov. 29, 1928.
The following methods are to be tested in various combinations: wet burlap, calcium chloride, sisalkraft, asphaltic emulsion, coal tar, Hunt process, tar paper, sodium silicate, and no curing.

Field Experiments in the Curing of Concrete Pavements.
Public Roads, v. 9, p. 139, Sept., 1928.
Cooperative tests conducted by Maryland State Roads Commission and the U. S. Bureau of Public Roads; results of cylinder and beam tests discussed.

Variation in Storage Conditions for Cement Test Specimens, by J. Keith.
Tonind. Ztg., v. 52, p. 808, 1928.
Effect of variation in air and water storage conditions upon results of strength tests.

Consistency of Silicate of Soda for Curing Concrete, by R. S. Beightler.
Eng. News-Rec., v. 100, p. 316, Feb. 23, 1928.
Results of tests on an experimental road in Ohio indicate that a more effective seal can be had with the same amount of sodium silicate by properly protecting the concrete until the day following its placing, instead of applying it to the green concrete.

Effect of Steam Treatment of Portland Cement Mortars on Their Resistance to Sulfate Action, by Thorvaldson and Vigfusson.
Eng. Jl., Mar., 1928.
Steam treatment resulted in greatly increased resistance to sulfate action, due primarily to the action of the steam on the aluminate in the cement.

On KL Storage, by H. Kühl.
Tonind. Ztg., v. 52, p. 1102, 1928.
The letters KL have no technical significance, but are the first and second letters of the word Kladno, the location of the laboratory of Gensbaur, who proposed this method.

Decrease in Strength of Portland Cement Briquets Stored Alternately in Air and Water, by Haegermann.
Tonind. Ztg., v. 52, p. 1101, 1928.
Discussion of the KL method of storage, which constitutes an additional 28-day water storage after the usual combined storage, which consists of 1 day in moist air, 6 days in water, and 21 days in air.

Curing-Tunnel Design, by W. D. M. Allan.
Concrete, v. 33, p. 17, July, 1928.
Principles of steam curing, and common defects in curing-tunnel designs.

Advanced Curing-Room Design.
Concrete, v. 33, p. 29, Dec., 1928.
Concrete Corporation, Milwaukee, builds additional curing rooms; doubles capacity of plant; mechanical regulation of temperature and humidity.

Destructive Agencies:

Damage to Cement Structures Caused by Dissolution or Disintegration and Outline of Means to Prevent Same, by R. Schlyter.
Pamphlet, Government Testing Station, Stockholm, Sweden, 1927.

Factors Affecting the Durability of Concrete, by F. R. McMillan.
Eng. Jl., Mar., 1928.

Conditions Necessary to Render Concrete Durable, by R. B. Young.
Eng. Jl., Mar., 1928.

Stone Preservation and Decay, by A. F. Laurie.
Jl. Roy. Inst. Architects, v. 35, p. 383, 1928.
Attention is called to mortar in joints, and salts introduced from subsoil water. Use of Si-ester and trass in mortar as stone preservative.

Preservation of Concrete, by Grün.
Bauing., v. 9, pp. 307, 350, 1928.
Causes of deterioration in concrete and methods of preserving against attack.

Deterioration of Concrete by Corrosive Waters, by R. Grün.
Chem. Fabr., pp. 281, 294, 1928.
Most damage is caused by H_2SO_4 either combined in sea water or free in natural acid water. Ordinary concrete can be made resistant by using a dense non-porous aggregate of uniformly sized material. At least 400 kg. of cement per sq. cm. must be used. Painting with bitumen is beneficial and better than covering with cement and clinker. Aluminous cement with 7 to 50% Al_2O_3 is very resistant to $MgSO_4$, but its behavior in presence of Na_2SO_3 is doubtful. The resistance of puzzolana to corrosion is very great, as is shown by the existence of water mains in good condition today, constructed by the Romans of this material.

Acid-Resistant Concrete, by Grigoriev.
Tonind. Ztg., v. 52, p. 175, 1928.
Portland cement may be mixed with acid-resistant material such as water glass, feldspar, grog, sand, trass, and red lead to form concretes somewhat resistant to acids.

Destruction of Concrete by Aggressive CO_2, by K. Biehl.
Zement, v. 17, p. 1102, 1928.
Water containing carbonate with a pH less than 7 is dangerous, while a value greater than 7 indicates that there is no danger of destruction, regardless of the total percentage of such material.

Reinforced Concrete in Gas Washer Structures, by Zimpell and Frank.
Gas und Wasserfach, v. 71, p. 952, Sept. 29, 1928.
Experience with corrosion of reinforced concrete columns in gas washer structure.

Earthquake Resistant Construction, by H. D. Newell.
Eng. News-Rec., v. 100, p. 650, Apr. 26, 1928, p. 699, May 3, 1928.
Natural period of vibration of buildings; Naito's methods of design; high office buildings; construction with flexible first story.

Test Light-Weight Aggregate Block and Concrete Floor Slab in Fire.
Concrete, v. 32, p. 29, Mar., 1928.
Haydite concrete block withstood heat 1480 deg.; reinforced concrete slab stood up well.

Behavior of Reinforced Concrete in a Big Fire, by P. Werker.
Bauing., v. 9, p. 322, 1928.
Most damage occurred where water was used to extinguish the fire and came in contact with the concrete; chemical extinguishers cause much less damage.

Fire Resistance of Sand-Lime and Concrete Brick Walls.
Tech. News Bull., Apr., 1928.
Eng. and Contr., v. 67, p. 326, June, 1928.
Materials and test specimens; method of testing; stability and load-carrying ability; fire effects; fire resistance classifications.

Significant Tests of Frost Action on Concrete, by W. H. Batchelder.
Eng. News-Rec., v. 101, p. 882, Dec. 13, 1928.
Tests indicate the importance of initial curing at high temperatures in cold weather concrete construction.

Effect of Low Temperatures and Freezing on Strength of High-Early Strength Portland Cement, by A. Gessner.
Zement, v. 17, p. 10, Jan. 5, 1928.

Resistance of Materials to Frost Action.
Tonind. Ztg., v. 52, p. 658, Apr. 21, 1928.

Effect of Low Temperatures on Strength of Cement, Mortar, and Concrete
Concrete and Const. Eng., v. 23, p. 574, Sept., 1928.
Results of tests carried out by Dr. Otto Graf from 1920 to 1926 at Stuttgart.

Some Accelerated Freezing and Thawing Tests on Concrete, by C. H. Scholer.
Proc. Am. Soc. Testing Mat., v. 28, 1928.
Eng. and Contr., v. 67, p. 455, Sept., 1928.
Method of testing durability of concrete by alternate freezing and thawing at low temperature; data are included covering field investigations of disintegrating concrete structures which show relation between field conditions and results of laboratory investigation by accelerated freezing tests; data include results on unsound aggregates, varying values of water-cement ratio, and variations in rate of freezing under different conditions of exposure.

Studies on Setting and Hardening of Concrete in Shafts Sunk by the Freezing Process, by Jungeblodt, Wesel, and Schmid.
Glueckauf (Essen), v. 64, p. 1337, Oct. 6, 1928.
Review of failures of concrete linings of shafts sunk by freezing process; results of laboratory experiments by Gruen and Werner at Duesseldorf and Ieverkusen; characteristics of concrete suitable for lining shafts sunk by freezing method.

Action of Pure Water on Various Hydraulic Cements, by P. Dumolard.
Rev. Mat. Const. et Trav. Pub., No. 224, p. 165, 1928.
Immunity of various concrete conduits to action of pure water due to an entirely physical characteristic of certain cements which enables the production of practically impermeable mortars.

Action of Distilled and River Water on Tensile Strength of Cement Test Briquets, by A. J. Blank.
Rock Products, v. 31, p. 66, July 7, 1928.
No retrogression in tensile strength of briquets stored in distilled water, regardless of kind of mixing water used, while opposite was true when river water was used for storage.

Action of Pure Water on Quick Setting Cements, by Mourral.
Le Genie Civil, v. 92, p. 121, 1928.
Pure water does not readily break down quick-setting cements used in water mains.

Sea-Water Resistant Cements, by G. J. Fertig.
Concrete (CMS), v. 33, p. 105, Sept., 1928.
Chemical factors which render cement and concrete more resistant to deterioration when exposed to sea water.

Reinforced Concrete—Its Permanence and Protection with Relation to the Action of Sea Water, by L. G. Frost.
Proc. La. Eng. Soc., v. 14, p. 156, Aug., 1928.
Causes of failure of concrete affected by sea water; methods for prevention of chemical disintegration.

Deterioration of Structures of Timber, Metal, and Concrete Exposed to Action of Sea Water, by Purser and Grose.
Bull. Dept. Sci. and Ind. Research, 1928.
Includes results of tests on reinforced concrete test block, with various depths of cover, exposed at Brisbane.

Marine Structures in Reinforced Concrete, by R. W. Stroyer.
Concrete and Const. Eng., v. 23, p. 338, May, 1928.
Concrete and Const. Eng., v. 23, p. 453, July, 1928.
Details of best mix; formwork; solid, hollow, and sheet piles; design and calculation of various forms of reinforcement.

Study of Reinforced Concrete Construction Near the Sea in Dutch East Indies, by C. Walterbeck.
Paper before International Congress for Testing Materials, Amsterdam, Sept., 1927.
Zement, v. 17, p. 65, Jan. 12, 1928.
Main cause of destruction is rusting of reinforcing steel and corresponding splitting of concrete. Salt water may have promoted action by supplying water for rusting, while a direct effect of salt is negligible. Permeable concrete gave steel opportunity to start corrosion.

Chemical Resistance of Cements, by C. Preussing.
Zement, v. 17, p. 383, 1928.
Review of paper by Grün, "Effect of Sea Water on Concrete" (*Zement*, v. 16, p. 1180, 1927), with regard to value of tests on mixtures of blast-furnace slags and Erz cement in sodium sulfate action.

Relative Resistance of Various Cements to Sulfate Waters, by T. Thorvaldson.
Eng. Jl., Mar., 1928.
Using high alumina, slag, natural, super, and portland cements.

Effect of Steam Treatment of Portland Cement Mortars on Their Resistance to Sulfate Action, by Thorvaldson and Vigfusson.
Eng. Jl., Mar., 1928.
Steam treatment results in greatly increased resistance to sulfate action due primarily to the action of steam on the aluminate in the cement.

Condition of Field Specimens of Concrete Exposed to Alkaline Soils and Waters Examined Dec., 1927, by C. J. Mackenzie.
Eng. Jl., Mar., 1928.

Resistance of Cement Concrete to Action of Sulfate Waters as Influenced by Cement, by D. G. Miller.
Proc. Am. Soc. Testing Mat., v. 28, 1928.
Tests on resistance of 30 portland cements to sulfate action. The most desirable cements for concrete exposed to action of sulfate waters are those that prove most resistant to the action both of pure salts and mixed salts.

Concrete Resists Sulfate Action, by D. F. Jennings.
Concrete, v. 32, p. 43, July, 1928.
Examination of reinforced concrete reservoir at waterworks at Dundee, Michigan, showed that quality of concrete was not affected after 10 years' service, although water supplied by wells has unusually high content of sulfates and sulfides.

Stability of Cements in Corrosive Waters, by Haegermann.
Pit and Quarry, v. 15, p. 67, 1928.
Density of concrete is an important factor as well as the nature of the aggregates and the after-treatment of the concrete.

Deterioration of Concrete, by W. Petry.
Tonind. Ztg., v. 52, p. 398, 1928.
Work of the committee on Concrete in Marsh Waters of the German Committee on Reinforced Concrete.

Oil Saturated Concrete, by O. Colberg.
Beton und Eisen, v. 27, p. 160, 1928.
After twenty years the concrete and its reinforcement were badly corroded, and the saturated concrete when heated to 285 deg. C. burned fiercely; discussion of means to prevent penetration of oil into concrete.

Efflorescence:

Efflorescence and Scumming of Mortar Materials, by H. Wilson.
Jl. Am. Ceramic Society, v. 11, p. 1, Jan., 1928.
Rock Products, v. 31, p. 47, Feb. 4, 1928.
Eng. and Contr., v. 67, p. 213, Apr., 1928.
Best preventive of mortar or wall efflorescence is absence of water since water is the only carrier of soluble salts; salts of barium to make the calcium insoluble are suggested remedy.

Kiln and Dry House Scum and Efflorescence in Face Brick Walls, by L. A. Palmer.
Proposed *Tech. Paper* Nat. Bureau of Standards, 1928.
32 types of brick examined; even when no mortar was used, but brick were partially submerged in distilled water, 17 developed efflorescence; thus, brick may be equally responsible with the mortars. An admixture of 2% ammonium or calcium stearate (by weight of the cement or lime) in the mortars will tend to reduce or prevent the extent to which mortar materials contribute to wall efflorescence.

Staining and Efflorescence on Indiana Limestone Caused by Moisture Seepage through Backing Masonry Materials, by Lee Huber.
Proc. Am. Soc. Testing Mat., v. 28, 1928.
Efflorescence on limestone usually consists of sulfates, carbonates, and chlorides of Ca, Mg, Na, and K. None of the 22 commercial waterproofers was entirely effective in preventing staining or efflorescence.

Efflorescence Observed on Hard Stone Facings on Either Side of Cement Joints.
Rev. Mat. Const. et Trav. Pub., No. 221, p. 14, 1928.

Flow under Load:

Flow of Concrete under Sustained Compressive Stress, by R. E. Davis.
Proc. Am. Concrete Inst., v. 24, p. 303, 1928.
Eng. and Contr., v. 67, p. 1925, Apr., 1928.
Plastic deformation under stress tested at University of California.

Fatigue of Concrete, by Mills and Dawson.
Proc. Highway Res. Board, v. 7, p. 160, 1928.
Results obtained in seven separate investigations. Values which have been obtained for the fatigue index as influenced by the rapidity of application of load and the period of rest between loadings are given. Various means of applying loads as developed by a number of individuals are discussed, as well as the effect of age on fatigue strength.

Jigging and Tamping:

Arthur Kill Bridges Paved with Joggled and Vibrated Concrete.
Eng. News-Rec., v. 101, p. 427, Sept. 20, 1928.
Vibrated forms and use of trap rock aggregate to obtain dense and strong concrete.

Devices for Tamping Concrete.
Eng. News-Rec., v. 101, p. 490, Sept. 27, 1928.
Use of electric tamper.

Lightweight Concrete:

Use of Lightweight Aggregate in Manufacture of Concrete Masonry Units, by A. W. Scheer.
Proc. Am. Concrete Inst., v. 24, p. 436, 1928.

Lightweight Aggregate Permits Erection of Two Additional Stories.
Concrete, v. 33, p. 13, Aug., 1928.
Data on haydite concrete.

Another Light-Weight Masonry Unit.
Concrete, v. 32, p. 30, Feb., 1928.
Bubblestone, weighing about 65 lb. per cu. ft., has compressive strength of about 250 lb. per sq. in. Denser mixtures have higher strength. Use a special air-cell with a 1:2½ mortar.

Lightweight Concrete.
Le Ciment, v. 33, p. 204, May, 1928.

Aerocrete Concrete, by A. Kleinlogel.
Beton und Eisen, v. 27, p. 213, June 5, 1928.

Aerocrete Roofs, G. V. Lang.
Concrete, v. 33, p. 42, July, 1928.

Reinforcement in Gas Concrete, by Meyer and Pukall.
Beton und Eisen, v. 27, p. 68, 1928.

Efficiency of Aluminum and Zinc as Gas Producing Media in Cement, by C. R. Platzmann.
Jl. Am. Ceramic Soc., v. 11, p. 211, Apr., 1928.

Manufacture of Porous Concrete by Action of Metallic Powders.
Le Genie Civil, v. 92, p. 521, May 26, 1928.

Porous Building Materials Made from Concrete, by J. Meyer.
Zement, v. 17, p. 510, 1928.
Concrete (CMS), v. 32, p. 108, May, 1928.
Discussion of methods of producing porous concrete, including the use of soap foam, gas-generating mixtures, and the "Schima" method in which powdered metal calcium is used to generate hydrogen.

Mixers and Mixing:

Mixing Time and Products Plant Economy, by E. G. Lantz.
Concrete, v. 33, p. 15, Dec., 1928.
Series of time of mixing tests conducted at Pittsburgh and Omaha.

Comparison between American and European Concrete Mixing Machines with Special Reference to A. B. G. Mixers.
Zeits. d. Oesterr. Ing. u. Arch. Ver., v. 79, p. 104, 1928.
Results of experiments.

Traveling Concrete Plants Line Cascade Tunnel.
Eng. News-Rec., v. 100, p. 224, Feb. 9, 1928.
Six mobile outfits, combining chutes, belt conveyor, concrete gun, and mixer, mix proportioned batches and place concrete.

Deaerated Concrete.
Canadian Eng., v. 54, p. 134, Jan. 10, 1928.
Mixing done in a vacuum. In a 1:2:4 mix made in the usual way, the concrete weighed 140 lb. per cu. ft., while by mixing the same materials by the deaerated method the weight was increased to 148 lb. per cu. ft. Compressive strength is increased by the increased density and the concrete is more waterproof.

Modulus of Elasticity:

One Hundred Fifty Years Advance in Structural Analysis, by H. M. Westergaard.
Proc. Am Soc. Civil Eng., v. 54, p. 993, Apr., 1928.

Elasticity of High-Grade Concrete Tensile and Compressive Strength Specimens, by B. Moehlmann.
Zement, v. 17, p. 390, 1928.

Extensibility of High Quality Concrete.
Pit and Quarry, v. 16, p. 85, Aug. 29, 1928.
Tests made using standard test pieces such as are usually made in testing tensile and compressive strengths.

Modulus of Elasticity of Cores from Concrete Roads, by A. N. Johnson.
Public Roads, v. 9, p. 164, Oct., 1928.
Results of determinations made on cores drilled from Maryland highways.

Determination of the Coefficient of Elasticity of Cement and Concrete from the Velocity of Propagation of Water-Hammer in Conduits, by De Sparre.
Rev. Gen. de l'Electricite, v. 26, Feb., 1927.
Le Ciment, v. 33, p. 29, Jan., 1928.

Measurement of Elastic Strains in Concrete Structures.
Tech. News Bull. No. 138, p. 144, Oct., 1928.
Test of practicability of a method which could be used in field tests for distinguishing between the elastic strains produced by stress and those resulting from changes in temperature and humidity, and from plastic flow or yield.

Extensometer for Determination of Young's Modulus for Concrete, by V. C. Davies.
Engineering, v. 125, p. 131, Feb. 3, 1928.

Pavements:

Design and Construction of Concrete Pavements, by C. Older.
Proc. Am. Soc. Civil Eng., v. 54, p. 147, Jan., 1928.
Rational computations of stresses; design treatment for joints and cracks; construction details.

Theory of Concrete Pavement Design, by H. M. Westergaard.
Proc. Highway Res. Board, v. 7, p. 175, 1928.

Design of Pavement Concrete by Water-Cement Ratio Method, by F. H. Jackson.
Public Roads, v. 9, p. 124, Aug., 1928.
Roads and Streets, v. 68, p. 479, Oct., 1928.
Discussion of water-cement ratio theory; determining ratio of fine to coarse aggregate.

Field Control of Pavement Concrete, by H. S. Mattimore.
Roads and Streets, v. 68, p. 209, Apr., 1928.
Paper presented before Michigan Conference on Highway Engineering.

Structural Design of Roads, by A. T. Goldbeck.
Am. Highways, v. 7, p. 4, Jan., 1928.
Design of various types of road construction.

Control and Special Mixtures for Road Concrete, by J. H. Chubb.
Eng. News-Rec., v. 100, Jan. 5, 1928.
Scientific design of concrete mixes for pavements; early strength concrete gives quick service.

Material and Construction Problems for Concrete Pavements, by G. Rüth.
Betonstrasse, p. 54, Mar., 1928.
Gives strength and shrinkage results for mortar specimens with various cements and sands.

Specifications for Concrete Pavements, by W. A. Maclachlan.
Canadian Eng., v. 55, p. 295, Oct. 9, 1928.
Comparison of practices and methods employed by various provinces and states in regard to fine and coarse aggregates, proportioning and mixing, consistency, cross-sections, forms, joints, finishing, curing, and reinforcement.

Requires Separate Size of Stone for Road Concrete, by R. T. Giles.
Eng. News-Rec., v. 100, p. 120, Jan. 12, 1928.
Segregation of stone in field handling and stock piling often offsets or nullifies effort at accurate control. For this reason, delivery on the job in different sizes and reproportioning at mixer is desirable.

California Job Shows Modern Methods of Concrete Pavement Construction.
Concrete, v. 32, p. 13, June, 1928.
Proportioning aggregates by weight or volume; design mix using three sizes of coarse aggregate; graded aggregates taken from commercial central batching plant; weakened plane joints; joint construction details; longitudinal float finishing.

Concrete Control on an Iowa Pavement Job.
Concrete, v. 33, p. 21, Oct., 1928.
Four sizes of aggregates used; materials measured by weight; water-cement ratio control; thickened edge on tops of pavement.

Laying Concrete Pavement at Port Alma.
Canadian Eng., v. 54, p. 109, Jan. 3, 1928.
Unique methods of construction and equipment.

Advanced Scientific Methods Used on Concrete Road Work in Tennessee, by R. H. Baker.
Concrete Highways and Pub. Imp., v. 12, p. 226, Oct., 1928.

Twelve-Mile Concrete Demonstration Road in Virginia, by A. C. Benkelman.
Roads and Streets, v. 68, p. 403, Aug., 1928.
Construction features.

Fourth Progress Report on the German Highway Commission's Test Road at Braunschweig.
Betonstrasse, p. 131, May, 1928.

Field Tests Permit Early Opening of Pavements, by F. J. Flood.
Concrete, v. 32, p. 41, Jan., 1928.
Factors that permit opening of highways to traffic at early ages.

Concrete Pavement Opened to Traffic at Age of 46 Hours.
Concrete, v. 32, p. 31, Jan., 1928.
Methods used to secure high early strength on pavement job in Wyandotte, Mich. The remarkable results were obtained by careful control of the water-cement ratio, careful selection of aggregates, and thorough mixing.

Concrete Paving-Base Control by Core Tests, by E. A. Kemmler.
Eng. News-Rec., v. 100, p. 661, Apr. 26, 1928.
Systematic core boring and core testing stabilize strength and slab depth in pavements.

Core Tests Lead to Proportioning by Weight, by C. E. Foster.
Eng. News-Rec., v. 101, p. 19, July 5, 1928.
Change from volume mixtures looking ahead to designing for given strength follows 2 years' core-drill work of Michigan State Highway Dept.

Reinforced Concrete Approach Slags for Highway Bridges, by W. H. Rabe.
Eng. News-Rec., v. 101, p. 352, Sept. 6, 1928.
Special practice adopted by Ohio Department of Highways to prevent roughness of road at ends of bridges.

Reinforcement for Concrete Roads, by R. A. B. Smith.
Concrete and Const. Eng., v. 23, p. 199, Feb., 1928.

New Concrete Pavement Has Removable Top Surface.
Eng. News-Rec., v. 100, p. 38, Jan. 5, 1928.
Splitting plane incorporated between top and base.

Machine for Testing Surface Smoothness.
National Sand and Gravel Bull., v. 9, p. 20, Apr., 1928.
Called the "bumpometer."

Instrument Developed for Measuring Cracks in Concrete Pavement, by H. L. Bosely.
Concrete, v. 32, p. 32, Apr., 1928.
Bureau of Roads secured accuracy of 0.25%; details of operation.

Traveling Shed Trains Shelter Both Workers and Fresh Concrete from Tropical Rains, by N. C. McCloud.
Concrete, v. 33, p. 22, Aug., 1928.
Device invented by road builders; travels on steel side forms; hauled by cables attached to paver.

Research in Portland Cement Concrete Pavements, by I. W. Teller.
Proc. Highway Research Board, v. 7, p. 157, 1928.

Recent Developments in Highway Research, by V. L. Glover.
Roads and Streets, v. 68, p. 105, Feb., 1928.
Paper presented at meeting of American Association of State Highway Officials.

Recent Developments in the Highway Construction and Maintenance Field.
Roads and Streets, v. 68, p. 75, Feb., 1928.
Abstracts of papers presented at the 1928 Convention of the American Road Builders' Association.

Important Developments during the Past Year in Highway Research, by A. C. Rose.
Roads and Streets, v. 68, p. 101, Feb., 1928.
Summary of reports presented at the annual meeting of the Highway Research Board.

Forty Years' Progress in Road Design and Construction, by A. W. Cross.
Surveyor (London). v. 73, p. 601, June 8, 1928.

Trend of Practice in Concrete Paving, by L. S. Trainor.
Canadian Eng., v. 55, p. 29, Oct. 9, 1928.
Proportions; mixing time; finishing; curing; reinforcement; joints; thickness and width.

Permeability and Waterproofing:

Determination of Permeability of Concrete, by I. L. Collier.
Proc. Am. Soc. Testing Mat., v. 28, 1928.
Concrete, v. 33, p. 35, Aug., 1928.
Eng. and Contr., v. 67, p. 469, Sept., 1928.
Flow of water, under 145 lb. per sq. in. pressure, through concrete. The tests showed (1) a general straight-line relationship between flow of water through concrete and the water-cement ratio; (2) flow decreased as the cement content was increased; and (3) flow decreased almost directly in proportion to increase in percentage of sand.

Permeability of Concrete, by W. Hugentobler.
Ber. Komm. Abdicht. Schwerz. Wasserwirt., No. 5, p. 96, 1928.
Beton und Eisen, v. 27, p. 261, 1928.
Permeability of concrete varies with the amount of cement and inversely with the amount of water. Stone dust and hydraulic lime have little effect, although up to 9% of hydrated lime is beneficial. Natural sand and gravel give better results than crushed rock. When specimens are stored damp, the permeability is greater than when stored dry, although the former on drying out approach the latter. The penetration of water is faster at the start, and depends upon the pressure. Specimens stored 53 and 97 days gave similar results. A pitch coating withstood fairly high pressures, while bituminous coatings were sometimes helpful, sometimes not. Surfaces plastered with cement mortar withstood small pressures, but not large ones. A metal coating sprayed by the Schoop process withstood 15 atm. pressure.

Surface Protection of Concrete against Deleterious Solutions, by H. Dubiel.
Zement, v. 17, p. 70, 1928.

Waterproofing Mortar and Concrete, by E. Marcotte.
Arts et Metiers, 1927.
Le Ciment, v. 33, p. 21, Jan., 1928.

Protective and Waterproofing Coatings for Concrete, Mortar, and Stone, by H. E. Schubert.
Bauing., v. 9, p. 325, 1928.
Composition and method of use of 9 materials for coating the surface of concrete, and the Contex process for surfacing in such a manner as to expose the aggregate.

Application of Protective Coatings to Concrete, by R. Grün.
Tonind. Ztg., v. 52, p. 824, 1928.
Concrete to be treated should contain not less than 450 lb. of cement per cu. yd. of concrete, an aggregate capable of producing a dense mortar is necessary, and an excess of mixing water must be avoided.

U. S. Government Master Specification for Integral Waterproofing Material for Use with Portland Cement Mortar or Concrete.
Circular No. 360, Nat. Bureau of Standards, 1928.

Integral Waterproofing Compounds for Concrete, by M. B. Lagaard.
Bull. 6, Univ. of Minn., Nov., 1927.
Tests on 12 waterproofing compounds and 2 waterproofing cements. All showed a reduction in strength except one, which showed only a slight increase. In some cases the concrete was consistently made more watertight, while in other cases the effect was only slight. In no case was workability improved.

Waterproofing Concrete Integrally, by J. A. Meacham.
Concrete, v. 33, p. 19, Nov., 1928.
A comparative test of well-known commercial methods and materials classified into the following groups: (1) use of portland cement and selected aggregates in richer and better balanced mixes, (2) admixtures consisting of finely divided, more or less inert materials, such as lime, diatomaceous earth, etc., intended to increase the density and permeability by reducing the amount of voids, (3) special cements which may be either the product of improved methods of manufacture, or ordinary portlands treated with waterproofing compounds at the mill, and (4) chemical compounds of a definitely water-repellent nature, which may result either from negative capillarity or colloidal action. The test results indicate that no product should be assumed to be satisfactory waterproofer until definitely proved so by test under conditions approximating those expected in the finished work.

Placing:

Placing Concrete under Water.
Concrete, v. 32, p. 42, Feb., 1928.
No leaner mix than a 1:2:4 should be used.

Experiences in Placing Poured Concrete under Water, by F. R. Habicht.
Beton und Eisen, v. 27, p. 60, 1928.
Results of tests show that the resulting concrete was more impervious and stronger than specimens obtained from dry mixes containing similar proportions of cement and aggregate and placed in air.

Belt Conveyors Place Concrete.
Concrete, v. 32, p. 21, June, 1928.
Concrete placed in 2700-ft. Seventh Street viaduct at Decatur, Ill., and in California flood control job taken from mixer to forms on portable belt conveyors.

Placing Concrete with Belt Conveyors.
Concrete, v. 32, p. 17, Mar., 1928.
Use of belts for conveying concrete from mixers or cars to the forms; operation data; precautions.

Belt Conveyor Places Concrete for New Maumee-Perrysburg Bridge, by S. A. Baxter.
Concrete, v. 33, p. 32, Oct., 1928.
300 cu. yd. placed per 10-hr. day.

Proportioning:

Design of Concrete (Report, Sub-Com. IV of A.S.T.M. Com. C-9).
Proc. Am. Soc. Testing Mat., v. 28, 1928.
Selection of cement, aggregates, and water; selection of strength for which to design; determination of proper proportions of fine and coarse aggregate to give desired workability for the given water-cement ratio; determination of water-cement ratio applicable to material selected; conversion of the proportions into field units.

Design of Concrete Mixtures, by P. E. Kressly.
Concrete, v. 33, p. 13, Dec., 1928.
Practical method of designing quality concrete; tables; charts.

An Exact Basis of Proportioning, by J. A. Kitts.
Concrete, v. 33, Sept., 1928.
Formulas are worked out to obtain an exact basis or proportioning based on the (1) uniform grading of aggregate particles, having a maximum and minimum size limitation; (2) absolute concrete; and (3) an absolute volume of mixing water to a unit volume of cement.

Tables of Quantities of Materials for Concrete.
Bull. 4, National Sand and Gravel Assn., May, 1928.

Cement Requirements for Some Concrete Mixes, by Stanton Walker.
Concrete, v. 32, p. 45, Apr., 1928.
Discussion of Harold Allen's article in Nov., 1927, issue and presentation of another method of determining the amount of cement required for different mixes, taking into account the amount of water used.

Concrete Production Control, by J. A. Kitts.
Concrete, v. 32, p. 13, May, 1928.
Factors determining strength, density, weight, wear resistance, workability, and pourability of concrete.

Systematic Proportioning of Mortars and Concretes, by R. Dutron.
Rev. Mat. Const., No. 220, p. 15, Jan., 1928.
A formula for calculating the water-cement ratio required to obtain a specified quality of concrete, given the absolute volumes of the cement, the aggregates, and the voids in the aggregate.

Graphical Presentation of Proper Proportioning and Predetermination of Compressive Strength of Mortar and Concrete by Means of the Parallelogram for Combination of 4 Materials.
Beton und Eisen, v. 27, p. 16, Jan. 5, 1928.
Detailed description of methods, giving examples and graphs—actually an extension of Feret's triangle.

Modern Methods of Designing Portland Cement Concrete, by W. G. Bragg.
Rock Products, v. 31, No. 10, p. 81, 1928.
Paper read before the Lehigh Mineral Industries Conference, Apr., 1928.

Quality of Cement in Concrete Mixes Specified by Volume Ratios, by J. Luhrs.
Bauing., v. 9, p. 564, 1928.
Table is given showing the differences in strength of 1:5 and 1:8 mixes resulting from cements of different specific gravities. Desire to reduce cost tends toward use of lighter cement and consequent reduction of strength, against which no protection is afforded by specification.

Inundation Method of Measuring Sand, by A. A. Levison.
Nat. Sand and Gravel Bull., v. 9, p. 19, Sept., 1928.

Practical Application of the Water-Cement Ratio Method of Controlling Concrete, by W. E. Hart.
Concrete, v. 32, p. 37, June, 1928.
Strength specifications demonstrated as practical; how to write a specification incorporating the water-cement ratio control method; revising building code requirements; economy of controlled concrete.

Water-Cement Ratio Concrete, by R. P. V. Marquardsen.
Eng. and Contr., v. 67, p. 313, June, 1928.
Formulas and forms based on field condition aggregates.

Comparison of Water-Cement Ratio Mixes and Ordinary Mixes, by C. F. Dingman.
Concrete, v. 32, p. 24, Jan., 1928.
Effect of workability requirements on amount of cement needed for cubic yard of concrete.

Practical Application of Water-Cement Ratio to Concrete Highway Pavement Construction, by W. F. Purrington.
Roads and Streets, v. 63, p. 189, Apr., 1928.
Results in New Hampshire described in paper presented before Boston Society of Civil Engineers.

Amount of Mixing Water for Concrete of Normal Consistency, by Luftschitz.
Zement, v. 17, p. 443, Mar. 15, 1928.

Control of Water in Concrete.
Canadian Eng., v. 54, p. 116, Jan. 3, 1928.
Description of apparatus.

Measurement of Concrete Materials, by R. T. Giles.
Canadian Eng., v. 54, p. 203, Feb. 7, 1928.

Proportion Aggregates by Weight in New Kansas City Products Plant, by C. E. Swanson.
Concrete, v. 33, p. 15, Aug., 1928.
Three kinds of aggregates combined by weight with traveling measuring hopper.

Premixed Aggregates for Concrete.
Canadian Eng., v. 54, p. 103, Jan. 3, 1928.
Condemnation of premixed aggregate because of excessive segregation.

Right Choice of Materials in Concrete Construction, by H. Fritzsch.
Zement, v. 17, p. 515, 1928.
Importance of sound knowledge of properties of raw materials and of methods and results of concrete testing and research; details of two failures which are attributed to the lack of such knowledge; description of Regulus mixer, which automatically mixes fine and coarse aggregate, cement, and water in any desired proportions.

Research-General:

Report of A.S.T.M. Committee E-9 on Correlation of Research.
Proc. Am. Soc. Testing Mat., 1928.

Examination of Mortar of an Excavated Roman Fort on the Rhine, by F. Dresler.
Tonind. Ztg., v. 52, p. 227, 1928.

Contribution to the Knowledge of Ancient Mortars, by K. Biehl.
Tonind. Ztg., v. 52, p. 346, 1928.
Structure of ancient German and Roman concretes described by means of photomicrographs.

Loriot and His Mortar, by F. Quietmeyer.
Zement, v. 17, p. 448, 1928.
Summary of rare memoir (1774) which shows that Loriot employed iron reinforcement in precast concrete block.

Mortar, by J. Worth.
Building Age, v. 50, p. 76, Nov., 1928.
Eight lime-portland cement mixes and four bricklayers' cements were tested for shear, tensile, and compressive strengths; recommended mortar, which contained twice as much lime as cement, was almost as strong as straight portland cement mortar in warm storage; cold weather did not decrease compressive strength of any specimens containing lime.

Mortar and Concrete, by E. Probst.
Pamphlet, reprinted from *Zement*, No. 23 and 24, 1928.

Sound Insulation:

Photo-Elastic Study for Acoustics of Buildings, by K. Slidell.
Eng. News-Rec., v. 100, p. 899, June 7, 1928.
Analysis of acoustical properties from plans and models prior to construction, permits of correction by modified design on use of sound-absorbent finish.

Sound Absorption of Cinder Concrete Building Units.
Bull. 1, Eng. Dept., Nat. Building Units Corp., 1928.
Comparison of tests made at University of Toronto and Detroit Testing Laboratory.

Applied Acoustics, by A. G. Huntley.
Arch. Jl., v. 68, p. 1749, 1928.
Sound absorption coefficients of wood, plaster, linoleum, acoustic plaster, artificial stone, and Cabot's quilt.

Strength Tests:

Tests of Large Concrete Cylinders.
Tech. News Bull., Bureau of Standards, No. 129, Jan., 1928.
Jl. Franklin Inst., v. 205, p. 249, Feb., 1928.
Tests on cylinders made during construction of Santeetlah Dam of 2, 3, 6, 8, 12, 18, 24, and 36-in. diameters; largest cylinder weighing 6300 lb. Stress strain readings were taken on 18 and 36-in. diameter specimens, readings being taken without stopping application of load. Ultimate strains varied from 0.0015 to 0.0032 in. per inch, and initial modulus of elasticity varied from 2,200,000 to 3,600,000 lb. per sq. in. In all cases the average modulus of 36-in. cylinders was higher than for 18-in. cylinders.

Effect of Physical Properties of Stone Used as Coarse Aggregate on the Wear and Compressive Strength of Concrete, by Thomas and Parkinson.
Bull. 2814, Univ. of Texas, Apr., 1928.
Compressive strength of concrete is affected by both the percentage of wear and compressive strength of the stone, although the effect of these two factors is not large except for stone showing high wear and low strength. Strength of concrete made from screenings as fine aggregate is less than for river sand, but the amount of wear for screenings is less than for sand. The highest strength is given by 1¼-in. stone, but minimum wear is found for 2-in. stone. In all cases, the two larger sizes show greater strength and greater resistance to wear than ¾-in. stone.

Study of Failure of Concrete under Combined Compressive Stresses, by Richart, Brantzaed, and Brown.
Bull. 185, Univ. of Ill. Eng. Exp. Sta., Nov. 20, 1928.
In general, the strength of mortar and concrete was as great in biaxial compression as in simple compression, and in a great many cases, it was greater. The strength of concrete in triaxial compression was found to increase greatly with the magnitude of the smallest principal stress.

Recent Experiments on Compressive and Bending Strengths, Contraction and Expansion, Abrasion Resistance, Permeability, and Chemical Resistance of Cement, Mortar, and Concrete, with Special Reference to the Effect of Grading of the Mortar Constituents, by O. Graf.

Beton und Eisen, v. 27, No. 13, p. 247, 1928.
Grading of cement and sand to obtain maximum compressive strength from any practicable mixing ratio depends not on the ratio but on the general shape of the sand particles. Addition of trass or Jura lime increases compressive strength, as does finely powdered clay if it is limited to 20%.

Strength of Cubes and Cylinders as the Basis of Concrete Testing, and the Safety of Plain and Reinforced Concrete Buildings, by W. Gehler.
Bauing., v. 9, pp. 21, 40, 63, 1928.
Effect of friction between the head of the testing machine and the specimen on the results obtained in compression tests on concrete cubes and cylinders.

Determination of the Compressive Strength of Mortar and Concrete, by Dutron.
Rev. Mat. Const. et Trav. Pub., No. 226, p. 254, July. 1928.

Effect of Shape and Character of Coarse Aggregate on Strength of Concrete, by F. C. Lang.
Concrete, v. 32, p. 37, Mar., 1928.
Tests of Minnesota Highway Department to determine behavior of variety of aggregates in concrete of constant water-cement ratio.

Effect of Amount, Size, and Grading of Coarse Aggregate on Compressive Strength of Concrete.
Tech. News Bull. No. 134, Nat. Bur. of Standards, p. 86. 1928.

Relation between Strength and Elasticity of Concrete in Tension and Compression, by J. W. Johnson.
Bull. 90, Iowa State Coll. Eng. Exp. Sta., May, 1928.
Modulus of elasticity in tension and compression; effect of age and consistency on strength and elasticity of portland cement concrete; relationship between tensile and compressive strength and modulus of rupture; strength and elasticity of concrete made with limestone; strength and elasticity of Lumnite cement concrete; general strength-modulus and elasticity relationships.

Compression, Flexure, and Tension Tests of Plain Concrete, by Gonnerman and Shuman.
Proc. Am. Soc. Testing Mat., v. 28, 1928.
Roads and Streets, v. 68, p. 420, Aug., 1928.
Compression tests were made on 6 by 12-in. cylinders, and flexural tests on 7 by 10 by 38-in. beams loaded at the third points of a 36-in. span. Tension tests were generally made on 6 by 18-in. cylinders; a few tests were made on cylinders of other lengths and diameters. Special grips were employed to hold the tension specimens during loading.

Adjustability of Early-Strength Concrete in Tension and Compression Specimens, by B. Mohlmann.
Zement, v. 17, pp. 390, 581, 619, 658, 691, 1137. 1928.
Age, kind of cement, consistency of concrete, type of aggregate, mix, curing, and size of test piece were factors considered in relation to the ability of concrete to adjust itself under test.

Effect of Particle Grading on Rate of Increase of Tensile Strength of Cement and Concrete, by Hedstrom and Werner.
Teknisk Tidskrift, v. 58, No. 23, p. 41, 1928.
Development of a formula expressing tensile strength of hardened cement as a function of the square of the quantity of hydrated cement, with the statement that the tensile strength attainable is inversely proportional to the square of the water-cement ratio.

Tensile Strength of Mortar in Brickwork.
Rock Products, v. 23, p. 63, Nov. 10, 1928.
Tests of adhesion of mortar to sand-lime brick has furnished an opportunity for comparing the tensile strength of the mortar in the brick with the strength of the same mortar in the form of a standard briquet. The results reported were obtained with a 1:1:6 cement-lime mortar in which various amounts of diatomaceous earth were substituted for equal parts of lime.

Beam Tests of Pavement Concrete Placed by Two Methods, by T. R. Beeman.
Eng. News-Rec., v. 101, p. 200, Aug. 9, 1928.
Slabs cut from pavements laid under identical conditions show slightly superior strength of standard concrete.

Field Beam Tests Check Strength of Georgia Pavements, by F. M. Garnett.
Highway Mag., v. 19, No. 6, p. 159, June, 1928.
Since pavements under use act in flexure rather than direct compression, Georgia is testing concrete for beam strength; apparatus and methods used; how test is made; cost of apparatus.

Effect of Several Mechanical Features of Testing on the Determination of Flexural Strength of Plain Concrete, by Willis and Wray.
Proc. Am. Soc. Testing Mat., v. 28, 1928.
Points out diversity in accepted methods for making flexural tests; relation between results by different methods and effect of such features as length of span, type of bearings, and rate of application of stress; results obtained with different methods vary widely; length of span and type of bearings affect results considerably, indicating need for standardization of test.

Concrete Beam Testing Machine, by A. T. Goldbeck.
Crushed Stone Jl., v. 4, p. 4, July–August, 1928.

Mortar Beam Tests, by D. O. Woolf.
Roads and Streets, v. 68, p. 378, July, 1928.
Public Works, v. 59, p. 316, Aug. 1, 1928.
Cantilever testing apparatus described.

Strength Characteristics of Concrete, by A. N. Johnson.
Public Roads, v. 9, p. 177, Nov., 1928.
Results of study of modulus of elasticity, effect of moisture on strength, and behavior under repeated loads.

Study of Methods of Testing Concrete in the Fields, by C. A. Wiepking.
Proc. Am. Concrete Inst., v. 24, p. 212, 1928.

Temperature Effects:

Comparison of Effect of High Temperatures on Concrete of High Alumina and Ordinary Portland Cements, by Miller and Faulkner.
Bull. 43, University of Washington Eng. Exp. Station, 1927.
Effect of heating on weight and strength; relation of loss in strength to loss in weight.

Thermal Conductivity of Concrete, by A. Hummel.
Bauing., v. 9, p. 528, No. 29, 1928.
Coefficient of thermal conductivity of concrete is not constant. Its value varies from 0.5 to 2.00 and is influenced by the moisture content, type of aggregate, and proportions in which the constituents of the concrete are present. The lower value should be used in calculating the stresses due to temperature gradients.

Relation between Strength of Portland Cement Mortar and Its Temperature at Time of Test, by Parkinson, Finch, and Hoff.

Bull. 2825, Univ. of Texas, July, 1928.

The purpose of this investigation was to determine the effect on strength of temperature of mortar specimens immediately prior to and at the time of testing, and whether mortar strength, modified by temperature change, would return to its former value as a result of additional normal storage. The same general phenomenon was evident in tension, compression, and cross-bending tests, each showing the marked effect of increasing temperature in decreasing strength. The tension specimens seemed much more affected than the compression, while the cross-bending specimens were affected somewhat more than the tension specimens. There was a tendency to recover strength when change in temperature was followed by a period of normal storage.

Test Methods:

Capping Device for Concrete Cylinders, by P. J. Freeman.

Eng. News-Rec., v. 101, p. 777, Nov. 22, 1928.

Quick hardening compound, baselite, poured hot will adhere to any specimen, and a perfect cap can be made and the specimen ready to test in ten minutes.

Volume Changes:

Shrinkage Due to Setting, by E. Schott.

Rock Products, v. 31, p. 50, May 26, 1928.

Paper presented before German Portland Cement Manufacturers, 1928.

Volume Change of Portland Cement as Affected by Commercial Composition and Aging, by A. H. White.

Proc. Am. Soc. Testing Mat., v. 28, 1928.

Summary of tests conducted during past 26 years. Cements relatively low in alumina and high in iron oxide show the lowest volume changes of material magnitude. Attention is called to possible danger of liberating free magnesia in burning process if percentage of lime is raised too high.

Elastic Stresses Developed in Arch Dams by Varying Conditions of Temperature and Shrinkage of Concrete, by Haegelen.

Annales des Ponts et Chauss., Nov. and Dec., 1927.

Le Ciment, v. 33, p. 198, May, 1928.

Contraction of Cements, by Cocagne and Matras.

Le Ciment, v. 33, p. 232, 1928.

Science and Industry, v. 12, p. 89, 1928.

Best results are secured by using as little water in the mix as possible followed by copious wetting after the cement has reached its set, by applying rather lean mixes, and probably by using sharp aggregates.

Wear:

Wear Tests of Concrete, by Scholer and Allen.

Bull. 20, Kansas State Coll., Feb. 15, 1928.

Concrete, v. 33, p. 43, Mar., 1928.

Concrete spheres 9 inches in diameter were tested in a paving-brick rattler. The French coefficient of the coarse aggregate does not materially affect the wear resistance of the concrete. Different brands of cement produce concretes of different wearing quality. Excessive mixing water affects resistance to wear in about the same proportion as it affects the strength. Replacing more than ⅛ of the portland cement with natural cement wili reduce both strength and resistance to wear in proportion to the amount of portland cement replaced.

REINFORCED CONCRETE

Arches:

Shell Arch of Market Hall in Frankfurt, A. N., by A. Kleinlogel.
Beton und Eisen, v. 27, p. 11, Jan. 5, 1928.
Discussion of shell arch construction methods with detailed description of series of tests on concrete arch.

Shell Roof on Reinforced Concrete According to the Kolb Construction Method, by R. Saliger.
Beton und Eisen, v. 27, p. 1, Jan. 5, 1928.
Description of buildings and method of design and construction with a test roof built for experimental purposes. Arch deformations were obtained by means of photographs; results showed very high degree of uniformity.

Design and Construction of a Skew Arch, by S. C. Hollister.
Proc. Am. Concrete Inst., v. 24, p. 371, 1928.

Beams:

Web Stresses in Reinforced Concrete Beams, by Richart and Larson.
Bull. 166, University of Illinois Eng. Exp. Sta.
Bull. 175, University of Illinois Eng. Exp. Sta.
Eng. and Contr., v. 67, p. 275, May, 1928.

Dimension Tables for T-Beams, by R. John.
Zement, v. 17, p. 28, Jan. 5, 1928.
Values for wide range in relation between thickness of plate and height of beam.

Reinforced Concrete Beams without Inclined Rods, by P. Grunberger.
Beton und Eisen, v. 27, p. 228, 1928.
Slots are left in the concrete over the column, and after it is set, straight rods are laid therein. The slots are then filled with fresh concrete, and precautions are taken to ensure satisfactory bonding between the masses of concrete. In this manner the necessary tensile strength can be obtained in that part of a continuous beam which is over a supporting column.

Ultimate Strength in Diagonal Tension of Reinforced Concrete Beams, by H. E. Pulver.
Concrete, v. 33, p. 47, Nov., 1928.
Formula for computing ultimate strength in diagonal tension of concrete beams; test results.

Distribution of Concreted Loads among Parallel Beams, by B. Enyedi.
Beton und Eisen, v. 27 p. 352, 1928.
Tabulation of all cases which may occur in practice.

Tests on Reinforced Concrete Beams, by R. Saliger.
Proc. Int. Congress for Bridge and Structural Eng., 1928.
The following conclusions are drawn: (1) quality of steel used for reinforcement has little effect on deflection or formation of cracks for equal stresses in reinforcement; (2) maximum load increases with elastic limit of steel; (3) bond stress increases as stresses in reinforcement become greater.

Shearing Strength of Reinforced Concrete Beams, by E. Probst.
Bauing, v. 9, pp. 202, 244, 1928.

Gunite and Concrete Encasement to Increase the Strength of Structural Steel, by Morris and Shank.
Bull. 137, Ohio State University, 1928.
Tests made in cooperation with the Fritz-Rumer-Cooke Company and the Pennsylvania Railroad Company. Badly corroded structural

beams can be restored and augmented by adding reinforced gunite or poured concrete. In no case was there any evidence of failure by bond. The ordinary transformed sections methods for computing reinforced concrete may be used. Vibration caused by traffic caused no trouble. Poured concrete encasement acted the same way as gunite. Encasing of columns by either gunite or poured concrete greatly stiffens them. Every encased column failed by the upsetting of the ends.

Bridges:

Loading Tests on a Reinforced Concrete Arch, by A. L. Gemeny.
Public Roads, v. 9, p. 185, Dec., 1928.

Largest Concrete Arch Bridge in the World.
Beton und Eisen, v. 27, p. 37, Jan. 20, 1928.
Short description of highway bridge across the Caille at Cruseilles. Arch built of plain concrete, and has span of 139.8 m. with rise of arch of 27.0 m. Plain concrete more economical than reinforced concrete. Largest allowed compressive stress 70 kg. per sq. cm. (about 1000 lb. per sq. in.) and lowest, 10 kg. per sq. cm. (140 lb. per sq. in.).

Concrete Bridge Construction on Curve.
Eng. News-Rec., v. 101, p. 4, July 5, 1928.
Cableway adapted to handling materials for 13-span bridge on sharp curve at Reading, Pa.

Method of Concrete Control and Some Test Results in Construction of Concrete Arch Bridges across the Mississippi River between St. Paul and Minneapolis, by C. R. Hansen.
Bull. Minn. Fed. of Arch. and Eng. Soc., v. 8, p. 21, July, 1928.
Design and control of mix; concreting plant and pouring.

New Type of Reinforced Concrete Bridge in India, by Mears and Thadan.
Concrete and Const. Eng., v. 23, p. 45, Jan., 1928.
Design and construction of bridge which will be entirely under water certain seasons of the year.

Highway Bridge Destruction Tests, by N. C. Cloud.
Concrete, v. 32, p. 34, Jan., 1928.
Unique highway bridge tests now under way in North Carolina made possible by destruction of concrete arch bridge.

Unusual Types of Reinforced Concrete Bridges, by H. F. Schwegler.
Concrete, v. 33, p. 44, Dec., 1928.
Design features of two interesting European bridges; piers included to shorten middle span; center span of continuous beam bridge over 50 feet.

Report of Tests Made on Swift Island Bridge over Yadkin River in North Carolina, by J. B. Hunley.
Jl. Am. Ry. Eng. Assn., v. 30, p. 67, July, 1928.
Bridge consists of three concrete arches of 140 ft. span with concrete girder approaches; application of test loads; measurement made on arch ribs.

Effect of Climatic Changes upon a Multiple Span Reinforced Concrete Arch Bridge, by W. M. Wilson.
Bull. 174, University of Illinois Eng. Exp. Sta., Feb. 14, 1928.
Extensive comparison of actual measured deformations with those theoretically computed for the Vermillion River Bridge, Danville, Illinois. For an open spandrel arch having a rib from 2 feet to 4 feet deep, the maximum probable variation in the mean temperature of the rib is about 90 deg. F. under the climatic conditions

existing in central Illinois, and the simultaneous temperatures within the arch do not vary by more than 20 deg. F. Moisture content of the concrete has no appreciable effect in the expansion and contraction when compared with that of temperature. The coefficient of temperature expansion was found to be 0.0000049 for the rib and 0.0000056 for the deck. The rise and fall of the crown checked within 10% of the theoretical values, maximum 0.62 in. The change in temperature caused the piers to rotate but did not produce appreciable stresses. The sum of the movements at the expansion joints was equal to the theoretical thermal expansion from end to end, but the values at each joint varied considerably from the expectations, and in some cases there was no movement noted.

Bunkers and Silos:

Reinforced Concrete Bunkers, by W. S. Gray.
Concrete and Const. Eng., v. 23, pp. 169, 243, Feb., Mar., 1928.

Design of Deep Circular Bins, by W. W. Hay.
Concrete, v. 32, p. 43, June, 1928.
Practical method for designing deep bins for storage of grain, cement, coal, and various rocks.

Precast Concrete Simplifies Imhoff Tank Work, by W. L. Couse.
Eng. News-Rec., v. 101, p. 430, Sept. 20, 1928.
Complex tank structure; difficulty of molding parts in place solved by precasting.

Calculation of Bending Stresses in Reinforced Concrete Tanks, by P. Pasternak.
Schz. Bauz., v. 90, pp. 241, 258, 267, 1928.

Chimneys:

Stress in Reinforced Concrete Chimneys, by Gillespie and Irwin.
Canadian Eng., v. 54, Jan. 24, 1928.
Charts, based on studies at Toronto University, for rapid approximate stress determination in both steel and concrete of reinforced concrete chimneys, due to weight, wind, and temperature changes.

Noranda's New Smelter Stack, by E. H. MacDermott.
Eng. and Mining Jl., v. 125, p. 649, Apr. 21, 1928.
Canadian Mining Jl., v. 49, p. 486, June 15, 1928.
Tallest reinforced concrete chimney in America.

Design and Construction of Reinforced Concrete Chimney Stacks, by A. Kleinlogel.
Beton und Eisen, v. 27, p. 189, 1928.
Heat stresses in chimney stacks.

Columns:

Tests on Spirally Reinforced Concrete Columns Having Cast-Iron Cores, by R. Saliger.
Beton und Eisen, v. 27, p. 329, 1928.
The permissible load for spirally reinforced concrete columns having cast-iron cores is given by the formula:
$N_z = (35 \text{ to } 45)F_k + 2000\ F_g + 1300\ F_s$, for square columns
$N_z = (35 \text{ to } 45)F_k + 2000\ F_g + 2000\ F_s$, for round columns
where N_z = permissible load in kg. per cm.^{2}; F_k = sectional area of concrete within the spiral reinforcement in cm.^{2}; F_g = sectional area of cast-iron core in cm.^{2}; F_s = area of spiral reinforcement in a horizontal section in cm.^{2}

Capping Structural Steel Cores in Reinforced Concrete Columns, by R. C. Reese.
Eng. News-Rec., v. 101, p. 593, 1928.

Dams:

Arch Dam Investigation of Engineering Foundation—Report by Committee. Published by American Soc. Civil Eng., May, 1928.

Approximate Formulas for Arch Dam Design, by B. F. Jakobsen.
Eng. and Contr., v. 67, p. 25, Jan., 1928.
Formulas for arch dam design; ordinary cylinder formulas unsatisfactory.

Analysis of Arch Dams by the Trial Load Method, by C. H. Howell.
Proc. Am. Soc. Civil Eng., v. 54, p. 61, Jan., 1928.
Tables, formulas, and curves.

Notes on Arched Gravity Dams, by Bauman, Cain, Hanna, Kramer, Wiley, Creager, Godfrey, and Turner.
Proc. Am. Soc. Civil Eng., v. 54, p. 219, Jan., 1928.

Baffle-Pier Experiments on Models of Pit River Dams, by E. S. Sherman.
Proc. Am. Soc. Civil Eng., v. 54, p. 363, Jan., 1928.

Designing a High Storage Dam for the Mokelumne Project, by F. W. Hanna.
Eng. News-Rec., v. 100, p. 444, Mar. 15, 1928.
Analysis of concrete arch dam.

Arch Dam Unusual in Design and Construction, by W. L. Scott.
Eng. News-Rec., v. 101, p. 325, Aug. 30, 1928.
Precast units used to control shrinkage and eliminate bending moments; all falsework based on cable support only.

Some Features of Testing the Stevenson Creek Dam, by W. A. Slater.
Proc. Am. Concrete Inst., v. 24, p. 273, 1928.

Building a Brick-Faced Concrete Arch Dam, by H. W. Reutershan.
Eng. News-Rec., v. 101, p. 268, Aug. 23, 1928.
Brick armor on both faces of Caneadea dam to resist frost action; difficult plant layout in deep gorge; cold weather concrete well protected.

New French Method of Building Concrete Dam.
Concrete, v. 32, p. 24, Mar., 1928.
Instead of a single wall, there are a series of walls, each comparatively thin.

Features of Design, Coolidge Multiple-Dome Dam, by C. R. Olberg.
Eng. News-Rec., v. 101, pp. 396, 438, Sept. 13 and 20, 1928.
Reinforced to resist temperature stresses; no contraction joints except in buttresses.

O'Shaughnessy Dam and Reservoir, by Gregory, Hoover, and Cornell.
Proc. Am. Soc. Civil Eng., v. 56, p. 405, Feb., 1928.
Dam of concrete masonry of gravity overflow type 1750 ft. in length. A reinforced concrete arch bridge of 12 spans crosses dam over spillway section.

St. Francis Dam Catastrophe—A Review Six Weeks After, by N. A. Bowers.
Eng. News-Rec., v. 100, p. 727, May 10, 1928.
More data on design and construction of the dam itself, the situation in the path of the flood down the valley, and abstracts of 4 reports of investigating committees.

Sixth Report on St. Francis Dam Offers New Theories.
Eng. News-Rec., v. 100, p. 895, June 7, 1928.
Abutments lifted by landslide at left bank and swelling under right: cracks permitted underscour.

Extremely Thin Dam Fails.
Eng. News-Rec., v. 101, p. 318, Aug. 30, 1928.
Failure of Nevada dam of unreinforced concrete, thickness at top only 18 inches, and at bottom not more than 36 inches; bottom of break was at construction joint at which no attempt had been made to bond concrete.

Notable Dam Failures of the Past.
Eng. News-Rec., p. 472, Mar. 22, 1928.
List of dams with reasons for failure.

Design:

Designing Reinforced Concrete Against Bending and Compression, by C. L. Christensen.
Eng. News-Rec., v. 101, p. 127, July 26, 1928.
Method of designing beams and columns permits direct calculation of steel without preliminary computations.

Application of Reinforced Concrete to Structures, by W. F. Zabriskie.
Concrete, v. 32, p. 39, Feb., 1928.

Materials and Workmanship for Reinforced Concrete.
Manual, Inst. of Struc. Eng., 1928.
Materials; proportioning; conditions as to quality, tests, rejection, welding, bends, and placing; formwork; bonding to hardened concrete.

Floors and Roofs:

Heavy Duty Concrete Floors, by C. E. Covell.
Concrete, v. 32, p. 26, May, 1928.
Methods of placing that will prevent dusting and undue wear.

Code Provisions for Concrete Floors.
Concrete, v. 33, p. 23, July, 1928.
Suggested formula for building codes: W is equal to $L - D/2 + 10$ in which W = reduced live load; L = specified load; D = deadweight of floor; but in all cases the reduced load may not be less than two-thirds the specified load.

Polishing Concrete Floors, by F. Grove-Palmer.
Carp. and Build., v. 102, p. 1140, 1928.
Treatment of China wood oil and transparent wood filler.

Frames:

Effect of Brackets in Reinforced Concrete Rigid Frames, by F. E. Richart.
Tech. News Bull., National Bureau of Standards, May, 1928.
Eng. and Contr., v. 67, p. 376, July, 1928.
Bureau of Standards *Journal of Research*, v. 1, p. 189, Aug., 1928.
United States Daily, v. 3, p. 1, Oct. 8, 1928.
Brackets can be used to effect a considerable saving of material and of dead weight.

Piles and Piers.

Pneumatic Caissons Sealed at Record Depths, Using Quick-Hardening Cement, by C. K. Allen.
Eng. News-Rec., v. 109, p. 484, Mar. 22, 1928.
Quick-hardening cement factor in rapid founding under air of Kennebec River bridge piers; caissons sealed at maximum water depths of 125 ft. on rough rock bottom.

Cast in Situ Concrete Piles.
Concrete and Const. Eng., v. 23, p. 205, Feb., 1928.
Reinforced Concrete Pier Piles.
Canadian Eng., v. 55, p. 209, Aug. 7, 1928.

Pipe:

Tests of Clay and Concrete Load-Bearing Pipe, by W. J. Schlick.
Proc. Am. Soc. Testing Mat., v. 28, 1928.
Discussion of problems of test procedure that are more commonly encountered; strength test; absorption test; freezing and thawing test.

Determination of Coefficients for Concrete Pipe from Actual Field Measurements, by F. C. Scobey.
Hydraulic Eng., v. 4, p. 491, Aug., 1928.
Coefficients for the formulas of Scobey, Williams-Hazen, and Kutter-Ganguillet.

Poles:

Armored Concrete Line Poles.
Canadian Eng., v. 54, p. 104, Jan. 3, 1928.
Comparison between use of wood and concrete poles for electrical transmission lines.

Reinforced Concrete Transmission Line Poles.
Concrete and Const. Eng., v. 23, p. 263, Mar., 1928.

Concrete Poles, by F. W. Bradshaw.
Concrete and Const. Eng., v. 23, p. 315, Apr., 1928.

Steel-Concrete Poles.
Concrete and Const. Eng., v. 23, p. 298, Apr., 1928.
Stobie concrete pole.

Concrete Poles.
Concrete and Const. Eng., v. 23, p. 595, Sept., 1928.

Slabs:

Design Tables for Reinforced Concrete Slabs Carrying Concentrated Loads, by R. Roll.
Bauing., v. 9, p. 615, Aug. 24, 1928.

Specifications:

Modern Building Regulations for Reinforced Concrete, by F. R. McMillan.
Eng. Jl., v. 11, p. 291, Apr., 1928.
Paper read before Toronto branch of the Engineering Institute of Canada, Mar. 15, 1928.

How Calculations Can Be Simplified by the Use of Charts, by W. F. Wiley.
Eng. and Contr., v. 67, p. 183, Apr., 1928.
Four short cuts in design for abutments and walls.

Proportioning Methods: Numerical Data and Numerical Examples, by B. Loser.
Published by Wilhelm Ernst and Son, 1927.
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American Concrete Institute Adopts New Concrete Code.
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Design and cost data; skew arch design; calculation of flat plates by the elastic web method.

New Russian Regulations for Reinforced Concrete.
Beton und Eisen, p. 373, Oct. 5, 1928.
Regulations for design and construction of reinforced concrete structures issued at Moscow in 1926.

Ties:

Experiment with Concrete Railroad Ties in Australia.
Rock Products, v. 31, p. 80, Jan. 7, 1928.
Melbourne Tramways Board uncovered a number of concrete sleepers which were laid in 1914 as an experiment. After 13 years' continuous service in the ground, they were found to be in as good condition as when first laid. Wood sleepers in a neighboring length of track had to be removed on account of deterioration. Concrete sleepers now considered for all future work.

The Development and Use of Cast Stone

By L. A. Falco*

The avowed purpose of the American Concrete Institute is "to provide a comradeship in finding the best ways to do concrete work of all kinds, and in spreading that knowledge." It might be presumed then that only strictly technical papers would have a place on the program of the convention, but it seems to me that at the opening of this session on cast stone we would do well to look back for a few minutes on the developments which have made cast stone a subject worthy of one entire session of the American Concrete Institute convention. Cast stone is one of the most refined forms of concrete. Its very nature as a medium of architectural expression demands that we pause a moment to consider whither we have come and whither we are going. It is, therefore, the purpose of this paper briefly to outline to you the developments and use of cast stone—its past achievements and future possibilities.

It is very gratifying to me to see that the American Concrete Institute has accepted the term "cast stone," which I believe has now finally attached itself definitely to the material of which I am about to speak. I have no doubt that the implication of artificiality and imitation suggested by some of its earlier names has been one of the influences which affected both its earlier and later developments.

It is within the memory of most of us here when artificial stone, as it was then generally known, was regarded from an esthetic standpoint as on a level with poured concrete and cement blocks, and was considered a useful commodity only when the cost of natural stone proved excessive. It had the same constituents as concrete and was in every sense merely concrete in the shape of stone. It was drab in color and uninteresting in texture. It had no enthusiastic advocates except perhaps the few manufacturers to whom we are deeply indebted for the pioneer work which they did.

Those men soon realized that their product must possess some greater intrinsic value if it was to attain any distinction and become a material suitable for better class buildings. At considerable more expense crushed marble and other aggregates of that class were obtained and used in place of trap rock, sand and gravel, then commonly in use. Along with this departure from usual practice, cutting to expose aggregates and the application of surface textures which were already known and understood as belonging to stone, such as bush hammering, were adopted. The

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improved appearance and its suitability for finer use was at once apparent and cast stone became a building material comparable with natural stone. Although there was some increase in cost accompanying these changes, cast stone now had an individuality which it had previously lacked and could logically claim consideration on a basis other than cheapness.

For a number of years cast stone experienced a normal steady growth. It achieved during those years a background of fine work which served to stimulate the interest of architects and builders, many of whom, however, were reluctant to leave established precedent. Nevertheless, cast stone found its way into important buildings of all kinds, such as churches, universities, mausoleums, municipal buildings, banks and office buildings. Many of these buildings stand today as equal to the best work of later years. Cast stone was by this time known and recognized as a building stone and no longer confused with cement blocks, garden furniture and other purely ornamental products.

The extraordinary increase in the use of cast stone in the past few years has been in line with the tremendous volume of building activity which we have witnessed since the war. The idea of cast stone as a substitute material has persisted among architects, however, and its economy in comparison with natural stone has been the largest factor contributing to this increase in volume.

There has been a vast amount of educational work done by individual manufacturers to uproot the fundamental misconception of the real field of cast stone. As opposed to this work and tending to neutralize the good that has been done, we find other manufacturers promoting the use of their goods through their skill in matching other stones, in many cases to the detriment of the better qualities of their own product. I want to take this opportunity to point out the harm which must naturally result to the industry through the confusion which counter policies of this kind create in the mind of those who buy and use cast stone.

It is obviously true that as long as the idea of substitution is allowed to continue to be associated with cast stone, just so long will it remain necessary to produce an article which is cheaper than the genuine. And to follow the same reasoning to its conclusion, as long as price is the controlling factor by which sales are made, just so long will the quality of the product be subordinated. I need not point out that both of these principles are contrary to good business policy. Nor do I need to prove the contention that cast stone can be made and has been made, commercially and at a profit, of a quality that makes the necessity of comparison with other stones absurd.

It is amazing to observe the progress which has been made in this industry when we consider that the aims and ideals of the manufacturers have been so varied. Architects have been and are still confused by the recommendations and practices followed by different manufacturers. It is unhappily a fact that in too many cases these recommendations are dictated by the expediencies of the case rather than by policies leading to the general upbuilding of the industry.

This general conception of cast stone as a substitute material has influenced the development of cast stone in other directions as well. In all established building materials there has come up through years of use a technique which is peculiar to each. This technique is developed either from the structural limitation or the natural and economical manufacturing processes. Appearance has, I think, been a secondary factor. In terra cotta, for example, there is the limitation of size imposed by the tendencies of clay to warp during baking, and work in this material can be easily recognized by the multiplication of small units. In granite, on the other hand, due to the more extensive application of hand work and the consequent increase in cost of producing it, we find a marked decrease in the number of joints and generally larger units in common use. Cast stone is at present subject to the standard practices of both and has practically no technique which can be called distinctively its own. It is possessed of certain individual characteristics, among which is the element of strength, and a practice should be built up around it embodying more fully this great natural asset. It is curious to note that, with the knowledge of reinforced concrete in its present advanced state, this tremendous advantage of cast stone over other competing materials has been so little employed.

I have tried to show that up to very recent years cast stone has been meeting a demand for a material similar to natural stone with a price advantage. I would like to show that vast new fields are opening for cast stone in which the natural, structural, and artistic advantages can be more fully utilized, and in which price advantage will be secondary.

We have entered a new architectural movement in this country which I will not attempt to define. The demands of this movement for new colors, new effects, and new uses of old materials are being answered by craftsmen of all trades. Cast stone is unquestionably finding its place in this new school of design. One of the outstanding characteristics of this new architecture is the elimination of many of the older forms of architectural treatment such as, cornices, band courses, pilasters, etc., and in their place the introduction of larger intricately moulded and ornamented surfaces. The cost of carving these surface enrichments from quarried stone would in many cases prohibit their use on many modern city buildings. They can readily be produced in cast stone without undue expense in large units and in a great variety of colors.

A great deal of architectural experimenting is being carried on in this new modernistic school. It would be regrettable if, during the evolution of this style, many of the established practices which have been inseparable with the use of stone were not abandoned in favor of others built around and developing the true economy of cast stone.

There is a present and insistent need for more educational work to be done. The architectural profession today demands, as never before, true and unbiased information on all materials which can be used in its designs and specifications. This information must come from trustworthy sources and have as its foundation the true ideals of service in

which the advantage of the individual manufacture is subordinate to the good of the work in question and the industry as a whole.

Technical associations have been the means selected by many industries to advance and encourage the use of their products. We have made our beginning and have formed the Association of Cast Stone Manufacturers. I feel certain that through it we can ally ourselves to the same aims and purposes so that in our relation with the architects we will be building a foundation of confidence and removing the cloud of confusion which now prevails. I believe we could, through educational work and exchange of ideas among ourselves, reduce the number of failures with which we are charged.

The basis of all improvements in the technique of manufacture and use must be careful research. To that end we have the investigations carried on by individual manufacturers, through fellowships in colleges and universities, by the U. S. Bureau of Standards and by the Portland Cement Association. By applying its own slogan to the field of cast stone the American Concrete Institute can do much for the benefit of users of this form of concrete. Through standard specifications on quality and through research carried on by its committees the Institute can lead cast stone manufacturers into that field of beauty and utility which the product merits.

The Physical Properties of Commercial Cast Stone*

By John Tucker, Jr., and G. W. Walker†

Introduction

As a result of a desire on the part of the Federal departments for a specification for cast stone, the Bureau of Standards has undertaken at the request of the Federal Specifications Board, the accumulation of data on the physical properties of commercial cast stone upon which such a specification may be written. Existing published data upon the properties of cast stone being very meager, the manufacturers and builders with whom the Bureau has been in contact are also keenly interested in having these properties determined. This paper presents some of the data which have been accumulated to date during this study and is in the nature of a progress report of an investigation in which much more remains to be done.

Source of Samples.—Several Federal departments generously turned over manufacturers' samples submitted for projected government work and a number of manufacturers very kindly submitted material especially for these tests. Pieces of cast stone were also obtained from several of the local jobs. The Bureau desires to acknowledge the cooperation of those manufacturers who submitted samples upon request and also of C. G. Walker of the Association of Cast Stone Manufacturers, who was helpful in securing a large number of samples. The origin of a number of the samples tested is unknown to the Bureau.

The properties of both the facing and backing material were determined where it was possible to cut such test specimens from the samples. From the structural point of view, the compressive strength and the modulus of rupture are the properties of most interest. Since, however, cast stone is not used as a load-bearing material the value of high strength would appear to lie in some possible correlation to the ability of the stone to withstand the disintegrating effects of weathering. The property of resisting weathering is of greater importance and if, as claimed by Prof. Kreuger[1] and others there is a correlation between resistance to freezing and absorption and porosity these latter properties are of importance and therefore have also been determined.

A portion of the data which have been secured in testing a large number of samples of commercial cast stone is presented at this time in

* Publication approved by the Director of the Bureau of Standards of the U. S. Department of Commerce.

† U. S. Bureau of Standards,

[1] *Transactions*, Royal Swedish Institute for Scientific and Industrial Research, No. 24, 1923

the hope that a discussion of it will suggest the types of requirements which should be placed in specifications for the purchase of cast stone. It is hoped, therefore, that the discussion center around these points, and also the following:

(1) While the data on absorption were obtained for the most part from specimens dried at 110 deg. C., it has been suggested that this

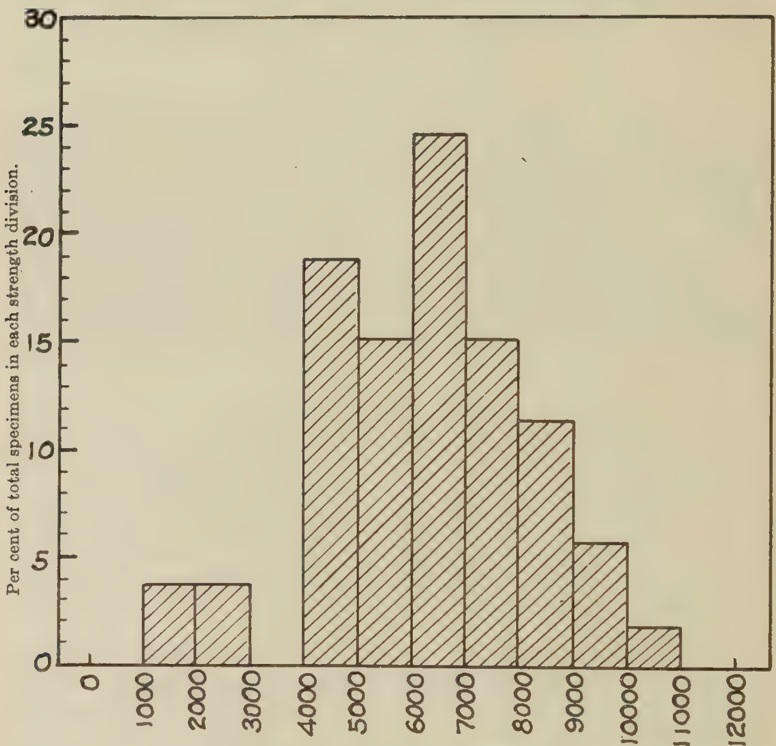

FIG. 1. RANGE OF COMPRESSIVE STRENGTHS OF CAST STONE SPECIMENS.

temperature is too high and that the drying should be done at a temperature of 65 deg. C., in dry air. We are making some further absorption determinations at this latter temperature but no data are now available. An expression of opinion of this suggested procedure would be particularly gratifying.

(2) We have been determining both the transverse strength and the compressive strength. The data obtained thus far do not show a close relation between these two strengths but we would like an expression of whether both of these determinations should be used or one, and which one.

(3) It is desirable to dry the specimens for strength tests after their preparation, since they must be cut from larger samples and by means which require flowing of water over the apparatus and specimens while the cutting is being done. The data presented on strengths have been obtained on specimens dried to constant weight at 110 deg. C. We have nothing to indicate what would have been the strengths had the specimens been dried at lower temperatures. Therefore, a discussion of the desira-

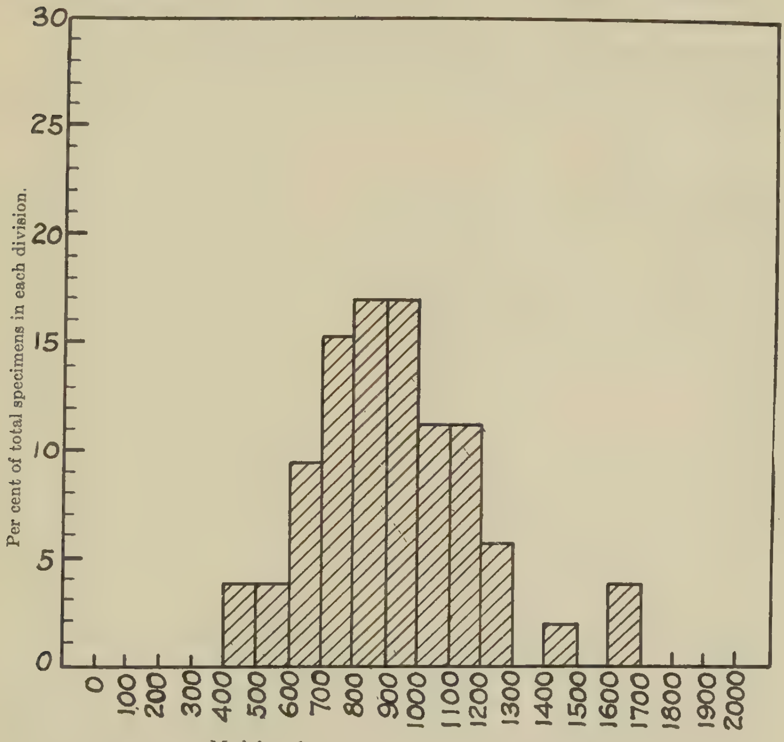

FIG. 2. RANGE OF MODULUS OF RUPTURE OF CAST STONE SPECIMENS.

bility of drying these specimens at lower temperatures is desired, as well as some expression as to whether others have used lower temperatures in drying for this purpose and, if so, what has been the effect on the strength.

(4) The freezing tests have not proceeded sufficiently far to permit of a general comparison of physical properties with freezing resistance but the results thus far obtained do not show a close relation between resistance to freezing and any one of the physical properties determined. The long time required for making freezing tests precludes the use of the

latter in specification requirements, and we would be pleased to know if anyone has obtained sufficient data and done anything towards correlating these properties for cast stone, and, if so, what is their expression of what should be the limits for the requirements.

DESCRIPTION OF TESTS

Selection of Specimens.—In the selection of the size and shape of the specimen to be used for the determination of the compression and weathering properties the following points were taken into consideration:

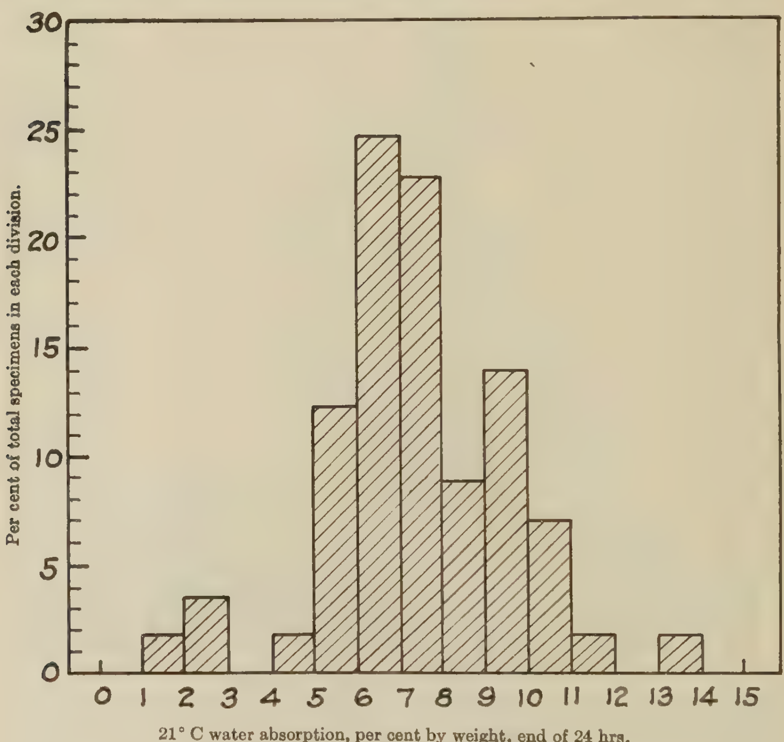

FIG. 3. RANGE OF ABSORPTION OF CAST STONE SPECIMENS.

(1) In the selection of a type of test specimen, the form should be such that the specimens can be cut easily and cheaply.

(2) Cores are to be desired rather than cubes or prisms because of the smaller investment required to set up a coring machine compared to a carborundum saw which is necessary in preparing rectangular specimens.

(3) On samples of well-made cast stone containing facing, a core, taken perpendicular to the face, longer than 2 in. would contain so much

of the backing that the compressive strength obtained would not be a fair measure of the strength of the facing material.

(4) In previous work at the Bureau on the determination of the compressive strength of limestone, marble, and other building stones cylinders 2 in. in dia. and approximately 2 in. long were used. A core of approximately the same size affords a comparison between the strengths of cast stone and natural stone.

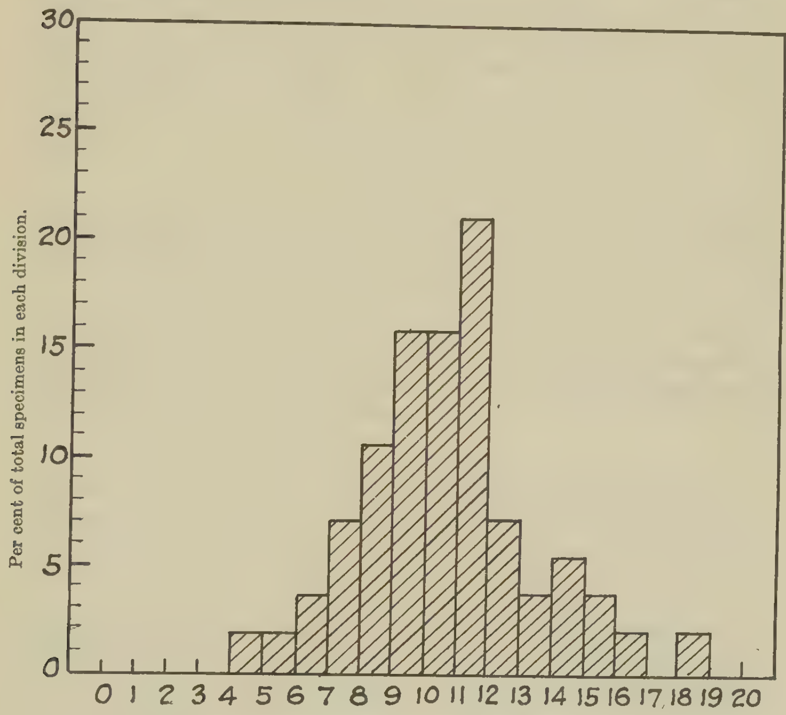

FIG. 4. RANGE OF BOILING ABSORPTION OF CAST STONE SPECIMENS.

(5) The diameter of a strength test specimen of mortar or concrete should be selected according to the largest size aggregate used. As this in the making of cast stone is seldom larger than that which would pass through a No. 4 sieve, a core of 2 in. dia. appears sufficiently large, as is corroborated by the relatively small variation in strength of similar specimens.

Taking these points into consideration a core 2 in. in dia. and 2 in. long was selected, to be used for the determination of the compressive strength and the resistance to weathering.

To obtain the modulus of rupture, or bending strength of the mate-

rial, a specimen of rectangular cross section is practically always used, and is most easily made. The specimen selected is 1 x 1 in. in cross section, and 8 in. long. In testing these specimens the supports were placed 6 in. apart, the load being applied at mid-span. One of the two portions of the broken specimens remaining from each transverse test was used for the determination of absorption.

Making the Specimens.—The sample of cast stone as received from the manufacturer was cut with a circular carborundum saw so as to obtain a piece slightly over 2 in. thick and large enough so that the cores and bars necessary for all tests could be cut from it. The coring machine used was a 3 spindle, belt drive, drill press which had been equipped with core drills made of seamless tubing and with automatic carborundum and water feed, thus making the cutting fairly rapid. The piece of cast stone was clamped to the base of the core machine and 6 cores cut from each specimen.

The 3 cores to be used for compressive tests were then finished by hand on a cast iron grinding lap with carborundum so that the ends were flat, parallel and 2 in. high, the maximum variation in height of specimen being $\frac{1}{16}$ in.

Three 1 x 1 x 8-in. prisms were cut from the sample with a circular carborundum saw and finished if necessary on the cast iron grinding lap. The maximum variation of cross sectional dimensions of the specimens was $\frac{1}{16}$ in. In computations of strength, these differences were corrected for.

METHODS OF TEST

Absorption Tests.—Three portions of the modulus of rupture specimens about 1 x 1 x 4 in., remaining after the cross bending tests, were dried at 110 deg. C. to constant weight. These were allowed to cool and then were totally immersed in distilled water at room temperature, very close to 21 deg. C. At the end of ½ hr., 2 hr., 4 hr., 6 hr., 24 hr. and 48 hr. of total immersion each piece was removed from the water, the excess water on the surface removed with a damp towel and the specimen weighed. From these data the weight of the water absorbed, expressed as the per cent of dry weight of the specimen, was calculated.

The specimens after 48 hr. cold water absorption were then placed in boiling water, boiled for 5 hr., cooled by the injection of cold water, and reweighed. The per cent of water absorbed by boiling may be considered as a rough measure of the maximum absorption of the stone.

Strength Tests.—The cores and bars after being cut were dried 48 hr. at 110 deg. C. and then cooled to room temperature in a desiccator where they were stored until tested. Compressive strength tests were made on a 50,000-lb. Riehle universal testing machine with the spherical bearing block below the specimen. The transverse tests were made on a machine developed by A. C. Harrison, formerly of the Bureau, using a rate of increase of load of approximately 140 lb. per min., the load being applied at the midpoint of a 6-in. span. The modulus of rupture was computed by means of the usual formula:

$$f = \frac{3\ Wl}{2\ bd^2}$$

where W = load, lb., applied at center of beam; l = clear span of beam (6 in. in these tests); b = breadth of beam, in.; d = depth of beam, in.

Apparent Porosity.—The volume of the water absorbed after the boiling of the specimens may be taken as a rough measure of the total pore space. The weight of the specimen suspended in water after boiling was measured and from this weight, together with the dry weight of the specimen and the weight after 5 hr. boiling, the per cent of the pore space was calculated, using the following formula:

$$P = 100\ \frac{W_b - W_d}{W_b - W_w}$$

where P = apparent porosity, per cent; W_b = weight of specimen after boiling; W_d = weight of specimen when dry; W_w = weight of saturated specimen suspended in water.

Freezing Tests.—Three of the cores 2 in. in diameter and 2 in. high were finished by hand but not with as great a precision as the compressive specimens. These were immersed in water for 48 hr. and then subjected to a series of cycles of freezing and thawing. The specimens were frozen while standing in ¼ in. of water so that any evaporation from the surface of the specimen may be given the opportunity of being replaced, at least in part by capillary action.

The specimens were frozen twice daily, 6 hr. during the day and 16 hr. at night. They were thawed in water at room temperature, warm water being added to compensate for the temperature drop of the water after the specimens were introduced. The minimum time of thawing was ½ hour.

The cores were examined at intervals of 5 cycles and the condition of the specimens noted. A record of observation was kept and the different types of disintegration noted. However, for the ease of reporting, two general stages of disintegration are shown in the table—(1) First signs of disintegration, and (2) the state of complete failure or where the structural value of the unit is destroyed.

The summary of the results of the tests is given in the accompanying table, which shows the mean values of the determinations and the lowest and highest values obtained.

In such a large table it is difficult to study the different values and to obtain an idea of the range of the several variables. To facilitate such a comparison the histograms given in Figs. 1 to 4 have been prepared. In these figures the percentage or proportion of the total number of test specimens that fall within certain limits have been given. Thus the distribution of these quantities (compressive strengths, etc.) which is of considerably more importance than the average value, is shown.

TABLE 1.—PROPERTIES OF SAMPLES OF COMMERCIAL CAST STONE

Reference Number	Description of Material and Thickness of Facing in Inches when not of One Material Throughout See Note (a)		Compressive Strength, lb. per sq. in. 2 x 2-in. Cylinders			Modulus of Rupture, lb. per sq. in. 1 x 1 x 8-in. Bar Tested on 6-in. Span			Absorption—Total Immersion (Per cent of water absorbed by dry weight of specimen, dried at 110 deg. C.) After Immersion of				Apparent Porosity, by Volume, per cent	Weathering (Alternate freezing and thawing) Total Cycles for	
			Facing Material	Face and Backing	Backing	Facing Material	Face and Backing	Backing	½ hr.	24 hr.	48 hr.	5 hr.[b]		First Sign	End Point
1	Gray Granite, 1¼-in. Facing	Average	....	6330	....	740	See note (b)	....	4.7	7.4	7.5	8.1	18.4	100[c]	105[c]
		Maximum	....	6610	....	740		....	4.8	7.4	7.5	8.1	18.5	...	...
		Minimum	....	6160	....	740		....	4 7	7.4	7.5	8.1	18.3	...	...
2	Gray Granite	Average	9130	....	....	1490	....	....	3.0	7.5	7.6	8.5	17.7	325	350
		Maximum	9350	....	....	1580	....	....	3.9	7.9	8.0	8.8	18.8	...	380
		Minimum	8910	....	....	1470	....	....	2.7	7.0	7.1	8.2	16.0	...	325
3	Gray Limestone	Average	8405	....	....	1190	....	....	3.2	6.9	7.0	7.3	16.4	125	130
		Maximum	8430	....	....	1370	....	....	3.7	7.1	7.2	7.4	16.6	...	135
		Minimum	8380	....	....	1050	....	....	2.9	6.8	6.9	7.2	16.3	...	125
4	Gray Granite	Average	8170	....	....	1100	....	....	2.5	7.6	7.7	9.2	20.0	500[a]	590[a]
		Maximum	8280	....	....	1160	....	....	2.7	7.9	8.0	9.4	20.4	500[2]	...
		Minimum	8100	....	....	1050	....	....	2.3	7.5	7.6	9.0	19.7	...	...
5	Buff Limestone	Average	8780	....	....	1290	....	....	5.3	7.2	7.3	8.4	17.6	370[a]	...
		Maximum	9400	....	....	1350	....	....	5.8	7.4	7.4	8.8	18.3	...	...
		Minimum	8050	....	....	1240	....	....	4.9	7.1	7.1	8.1	16.9	...	...
6	Gray-Brown Sandstone	Average	4720	....	....	900	....	....	1.7	4.0	5.0	11.1	22.7	140	200
		Maximum	4780	....	....	990	....	....	2.4	4.6	5.5	12.3	24.8	175	350
		Minimum	4660	....	....	720	....	....	0.9	2.9	4.4	10.1	21.2	90	150
7	Pink Sandstone, ½-in. Facing	Average	....	3780	....	....	1230	....	2.2	5.0	5.9	7.8	17.2	390	570
		Maximum	....	3970	....	....	1370	....	3.2	6.2	6.4	8.8	18.9	...	...
		Minimum	....	3590	....	....	1050	....	1.7	5.6	5.7	7.3	16.0	...	...
8	Gray Granite, 2-in. Facing	Average	4140	....	6000	870	900	800	3.7	6.8	7.1	10.4	19.1	370[a]	...
		Maximum	4220	....	6220	1030	1000	950	3.9	7.0	7.3	10.8	19.1	...	...
		Minimum	4060	....	5780	710	810	670	3.4	6.6	6.9	9.9	19.1	120	370[a]

Table 1.—Continued

9	Pink Granite, 1½-in. Facing........	Average	....	6560	7540	....	680	840	3.3	7.5	7.6	10.0	20.6	380	390[a]
		Maximum	....	6630	7690	....	690	910	3.5	7.5	7.6	10.4	20.6	...	...
		Minimum	....	6490	7390	....	670	770	3.0	7.4	7.6	9.7	20.6	...	...
10	Buff Limestone, 1-in. Facing	Average	....	....	6620	790	....	620	4.5	9.4	9.7	13.5	25.8	370[a]	...
		Maximum	....	....	6840	950	....	670	5.4	9.5	9.8	14.3	26.4	...	...
		Minimum	....	....	6400	660	....	580	4.1	9.3	9.6	13.0	25.2	...	...
11	Cream Sandstone	Average	7950	....	....	1080	....	....	2.2	5.5	6.1	11.2	23.6	100	150
		Maximum	8220	....	....	1210	....	....	2.5	6.0	6.7	11.6	23.7	...	190
		Minimum	7440	....	....	950	....	....	1.8	4.9	5.6	10.9	22.6	...	120
12	Gray Granite, 1½-in. Facing	Average	8950	....	....	930	....	....	0.9	2.6	3.6	8.1	17.5	95	180
		Maximum	10300	....	....	1110	....	....	1.0	2.8	3.9	8.7	17.9	...	220
		Minimum	8080	....	....	810	....	....	0.8	2.4	3.4	7.7	17.3	...	140
13	Cream Sandstone	Average	9170	....	....	1060	....	....	2.2	5.3	6.0	11.3	23.3	70	330[a]
		Maximum	9600	....	....	1140	....	....	2.4	5.6	6.2	11.9	24.2	80	...
		Minimum	8370	....	....	940	....	....	2.0	4.9	5.6	10.8	22.6	60	...
14	Light Gray Sandstone	Average	8790	....	....	960	....	....	3.8	5.3	5.4	7.5	16.7	...	380[a]
		Maximum	9150	....	....	1040	....	....	3.8	5.4	5.6	7.8	17.2	370	...
		Minimum	8350	....	....	950	....	....	3.7	5.2	5.3	7.4	16.3	...	..
15	Gray Sandstone	Average	7540	....	....	1170	....	....	5.8	6.6	6.7	9.9	20.8	240	370[a]
		Maximum	8360	....	....	1180	....	....	5.9	6.7	6.8	10.2	21.3	...	...
		Minimum	6430	....	....	1160	....	....	5.6	6.6	6.7	9.6	20.0	...	...
16	Buff Limestone	Average	5780	....	....	750	....	....	9.0	10.1	10.4	13.0	25.6	105	220
		Maximum	6020	....	....	770	....	....	9.3	10.5	10.8	13.4	26.2	120	...
		Minimum	5610	....	....	750	....	....	8.6	10.0	10.3	12.6	25.0	95	...
17	Light Gray Sandstone	Average	5260	....	....	900	....	....	5.7	6.8	7.0	11.0	21.1	120	200
		Maximum	5570	....	....	950	....	....	6.7	7.6	8.0	11.6	23.7	...	260
		Minimum	4680	....	....	870	....	....	4.5	6.0	6.3	10.4	17.8	...	140
18	Buff Limestone	Average	2430	....	....	560	....	....	8.4	9.2	9.5	14.3	27.7	100	210
		Maximum	2540	....	....	600	....	....	8.5	9.5	9.7	14.6	28.0	...	220
		Minimum	2380	....	....	520	....	....	8.3	9.0	9.3	14.2	27.5	...	200
19	White Sandstone, ½-in. Facing	Average	....	8190	....	....	820	....	3.5	5.5	5.9	9.6	21.0	105	220
		Maximum	....	8830	....	....	850	....	3.9	5.8	6.5	10.6	23.1	120	...
		Minimum	....	7800	....	....	800	....	2.7	5.0	5.4	8.6	19.3	100	...

TABLE 1.—CONTINUED

20	Gray Granite	Average	5980	....	....	600	....	....	4.7	6.9	7.1	9.6	21.2	210	325
		Maximum	6610	....	....	680	....	....	5.0	7.1	7.2	9.8	21.5	220	...
		Minimum	4920	....	....	510	....	....	4.4	6.9	6.9	9.5	21.0	200	...
21	Gray Granite	Average	4740	....	....	750	....	....	4 5	7.2	7.3	9.2	20.5	175	200
		Maximum	5050	....	....	800	....	....	4.6	7.4	7.5	9.5	20.6	...	...
		Minimum	4470	....	....	680	....	....	4.4	6.9	7.0	8.9	20.3	...	...
22	Red Granite	Average	5430	....	....	550	....	....	7.1	8.0	8.0	9.3	20.3	200	330[a]
		Maximum	5830	....	....	560	....	....	7.3	8.4	8.5	9.8	20.8	...	...
		Minimum	5120	....	....	530	....	....	7.0	7.7	7.8	9.0	19.9	...	...
23	Light Gray Marble	Average	6720	....	....	730	....	....	3.1	7.0	7.1	10.0	21.5	200	330[a]
		Maximum	6810	....	....	820	....	....	3.6	7.5	7.6	10.3	21.9	...	...
		Minimum	6600	....	....	650	....	....	2 7	6.5	6.6	9.6	21.0	...	...
24	Buff Limestone	Average	4410	....	....	770	....	....	9.5	11.6	11.7	14.6	28.7	175	330[a]
		Maximum	4500	....	....	820	....	....	10.0	11.9	12.1	14.8	29.0	205	...
		Minimum	4360	....	....	670	....	....	9.1	11.5	11.6	14.3	28.3	105	...
25	Buff Limestone	Average	4850	....	....	1030	....	....	6.5	9.8	9.9	12.5	25.0	70	200
		Maximum	5520	....	....	1130	....	....	6.6	9.9	10.1	12.6	25.2	80	...
		Minimum	3950	....	....	850	....	....	6.4	9.6	9.7	12.4	25.0	60	...
26	White Marble	Average	6560	....	....	840	....	....	4.9	6.6	6.8	12.1	25.4	110	130
		Maximum	7600	....	....	900	....	....	6.4	7.7	7.8	12.6	26.3	115	155
		Minimum	6200	....	....	770	....	....	4.0	5.9	6.2	11.4	24.2	90	95
27	White Marble, ½-in. Facing	Average	....	4470	....	....	220	....	9 0	9.8	10.2	15 6	29.2	85	150
		Maximum	....	4950	....	....	270	....	10 4	11 2	11 7	16.9	30 5	...	170
		Minimum	....	4090	....	....	180	....	7.6	8.2	8.6	13.6	26.7	...	130
28	White Marble, 1½-in. Facing	Average	....	4320	....	480	....	....	6 8	8 8	9 2	15.6	29 3	50	160
		Maximum	....	4580	....	520	....	....	8 5	9 9	10.2	16.9	31.0	...	...
		Minimum	....	3760	....	430	....	....	4 6	7.8	8.1	14.9	28.4	...	...
29	White Sandstone, 1½-in. Facing	Average	6630	....	....	890	....	....	4 7	6.8	7.1	11.9	25.2	170[1]	...
		Maximum	6880	....	....	1180	....	....	6 2	7.5	7.8	12.6	26.6	240[a]	...
		Minimum	6160	....	....	700	....	....	3.3	5.8	6.1	10.6	23.1	240[1]	...
30	Gray Granite	Average	5640	....	....	690	....	....	8 1	8.6	8.7	9.5	20 6	50	155
		Maximum	6380	....	....	780	....	....	8.3	8.8	8.8	9.7	20.9	...	...
		Minimum	5110	..	....	630	....	....	7 9	8.3	8.3	9.2	20 0	...	..

TABLE 1.—CONTINUED

31	Gray Limestone	Average	4390	....	....	880	....	....	9.3	10.4	10.6	11.2	23.4	...	60
		Maximum	4620	....	....	980	....	....	9.5	10.7	10.8	11.4	23.7	...	...
		Minimum	4140	....	....	770	....	....	9.1	10.2	10.4	10.9	22.8	...	...
32	Gray Granite	Average	7760	....	....	640	....	....	8.4	9.0	9.1	10.8	22 0	115	240[a]
		Maximum	8830	....	....	700	....	....	8.7	9.2	9.2	11.0	22.2	...	...
		Minimum	6050	....	....	590	....	....	7 9	8.7	8.8	10.6	21.6	...	...
33	Gray Granite	Average	6930	....	....	1070	....	....	7.4	8.2	8.2	8.4	18.6	55	100
		Maximum	7900	....	....	1120	....	....	7.5	8.3	8.3	8.6	19.1	...	105
		Minimum	6200	....	....	960	....	....	7.3	8.1	8.1	8.2	17.7	...	95
34	Buff Limestone	Average	7320	....	....	1270	....	....	3.7	5.8	5.9	10.0	21.2	65	180[a]
		Maximum	8100	....	....	1350	....	....	4.3	6.1	6.3	10.8	22.5	...	...
		Minimum	6020	....	....	1190	....	....	2.5	5.2	5.3	8.6	19.0	...	...
35	White Marble, 2-in. Facing	Average	4860	....	....	890	....	....	7.0	8 0	8.2	13.4	22.1	50	175[1]
		Maximum	5020	....	....	900	....	....	8.3	9.0	9.1	14.3	23.5	...	180[a]
		Minimum	4650	....	....	880	....	....	5.7	7.4	7.6	11.4	20.9	...	...
36	White Sandstone	Average	7730	....	....	800	....	....	5.2	6.3	6.5	10.1	21.7	50	180[a]
		Maximum	7960	....	....	840	....	....	6.7	7.2	7.4	11.2	23.0	...	...
		Minimum	7440	....	....	780	....	....	4.0	5.6	5.9	9.9	20.8	...	...
37	Light Gray Limestone, 2-in. Facing	Average	6880	....		850	....	....	5.1	6.8	7.2	11.6	23.6	50	180[a]
		Maximum	7150	....		900	....	....	7.4	8.5	8.8	12.7	25.7	...	...
		Minimum	6590	....	...	790	....	....	2.7	5.0	5.5	10.5	22.0	...	...
38	Gray Limestone	Average	5200	....	....	680	....	....	8.3	9.6	9.8	12.9	27.6	50	150
		Maximum	5530	....	....	760	....	....	10 5	12.0	12.3	16.9	32.7	...	170
		Minimum	4980	....	....	610	....	....	5.8	7.5	7.8	10.0	24.1	...	105
39	Light Gray Limestone	Average	6320	....	....	790	....	....	6.6	6.8	6.8	10.6	22.1	70[1]	...
		Maximum	6950	....	....	870	....	....	6.6	6.8	6.9	10.7	22.4	125[a]	...
		Minimum	5680	....	....	750	....	....	6.4	6.7	6.8	10.5	21.8	...	...
40	Buff Limestone	Average	8600	....	....	1230	....	....	5.7	6.5	6.5	6.5	14.5	180[a]	...
		Maximum	9050	....	....	1300	....	....	5.9	6.6	6.6	6.7	14.7	...	...
		Minimum	8180	....	....	1160	....	....	5.4	6.3	6.3	6.3	14.2	...	...
41	Gray Granite	Average	7770	....	....	960	....	....	6.1	7.0	7.0	7.1	16.1	50	180[a]
		Maximum	7980	....	....	1000	....	....	6.5	7.4	7.5	7.6	16.9	...	...
		Minimum	7610	....	....	920	....	....	5.9	6.1	6.8	6 8	15.9	...	...

TABLE 1.—CONTINUED

42	Light Gray Limestone	Average	5740	....	....	1000	....	....	9.7	10.4	10.6	11.9	24.6	160	325[a]
		Maximum	6080	....	....	1000	....	....	9.8	10.5	10.7	12.2	25.1	200	...
		Minimum	5270	....	....	980	....	....	9.7	10.4	10.5	11.5	24.4	120	...
43	White Marble	Average	5790	....	....	990	....	....	4.1	8.3	8.7	10.5	22.8	200	325[a]
		Maximum	6210	....	....	1040	....	....	4.5	8.6	9.4	10.7	23.2	...	...
		Minimum	5840	....	....	910	....	....	3.8	8.0	8.3	10.4	22.6	...	...
44	Gray Sandstone	Average	1870	....	....	450	....	....	12.3	13.0	13.6	18.2	32.8	25	65
		Maximum	2060	....	....	520	....	....	13.1	13.9	14.0	19.2	33.8	40	...
		Minimum	1760	....	....	380	....	....	11.4	12.0	12.2	17.3	32.1	12	...
45	White Marble	Average	6540	....	....	1010	....	....	2.9	5.5	6.0	9.6	20.6	55[2]	...
		Maximum	6600	....	....	1180	....	....	3.3	5.7	6.2	9.9	21.0	90[a]	...
		Minimum	6430	....	....	900	....	....	2.5	5.3	5.8	9.4	20.2	...	...
46	Light Gray Marble	Average	6170	....	....	1110	....	....	3.4	6.1	6.3	9.8	20.8	55[2]	100[a]
		Maximum	6270	....	....	1680	....	....	4.7	6.5	6.9	11.5	23.7	100[1]	...
		Minimum	6050	....	....	810	....	....	2.2	5.3	5.6	7.2	16.1	...	...
47	Buff Limestone	Average	6060	....	....	850	....	....	7.1	7.2	7.4	11.4	23.2	90[a]	...
		Maximum	6370	....	....	910	....	....	7.9	8.1	8.3	12.8	25.3	...	...
		Minimum	5830	....	....	760	....	....	6.6	6.6	6.8	10.5	21.7	...	...
48	Pink Limestone	Average	7060	....	....	780	....	....	7.0	7.1	7.3	11.4	23.2	90[a]	...
		Maximum	8110	....	....	1000	....	....	7.6	7.7	7.8	12.3	24.5	...	...
		Minimum	6250	....	....	410	....	....	6.0	6.2	6.3	10.1	21.1	...	...
49	Pink Granite	Average	7230	....	....	840	....	....	7.1	7.2	7.3	11.5	23.4	90[a]	...
		Maximum	7770	....	....	1080	....	....	7.5	7.7	7.8	12.3	24.6	100[1]	...
		Minimum	6910	....	....	730	....	....	6.3	6.5	6.5	10.4	21.7	...	...
50	Gray Granite	Average	6120	....	....	930	....	....	7.8	7.9	8.1	12.5	25.0	90[a]	...
		Maximum	6520	....	....	1160	....	....	8.9	9.0	9.1	14.1	27.3	...	...
		Minimum	5750	....	....	740	....	....	6.3	6.5	6.5	10.5	21.9	...	...
51	White Marble, ⅜-in. Facing	Average	....	3180	....	630	....	....	8.9	10.2	10.2	16.9	31.6	30	70
		Maximum	....	3520	....	680	....	....	9.1	10.6	10.6	17.5	32.6	35	...
		Minimum	....	2870	....	600	....	....	8.5	9.5	9.6	16.3	30.9	15	...
52	Buff Limestone	Average	9220	....	....	1620	....	....	1.0	2.4	2.8	5.8	13.1	50[a]	...
		Maximum	9640	....	....	1640	....	....	1.1	2.8	3.2	7.3	16.1	...	...
		Minimum	8630	....	....	1580	....	....	0.7	1.9	2.2	4.0	9.3	..	...

TABLE 1.—CONTINUED

53	Gray Marble	Average	10570	....	....	1640	....	....	0.7	1.6	2.0	4.1	10.8	55	60[a]
		Maximum	10840	....	....	1720	....	....	0.9	2.0	2.4	4.8	12.7	...	...
		Minimum	10260	....	....	1530	....	....	0.5	1.3	1.7	3.2	8.8	...	...
54	Light Gray Limestone	Average	4420	....	....	960	....	....	5.5	9.6	9.8	11.0	22.4	50[a]	...
		Maximum	4820	....	....	1170	....	....	5.8	10.1	10.3	12.1	24.2	...	...
		Minimum	4120	....	....	820	....	....	5.0	8.9	9.1	9.3	19.4	...	...
55	Buff Sandstone	Average	4660	....	....	970	....	....	9.1	9.1	9.2	14.1	27.2	70[a]	...
		Maximum	4670	....	....	980	....	....	9.5	9.7	9.8	14.4	27.8	...	...
		Minimum	4650	....	....	940	....	....	8.7	8.7	8.7	13.8	26.9	...	...
56	Gray Granite	Average	6790	....	....	1180	....	....	5.6	6.6	6.6	6.8	15.9	45	50[a]
		Maximum	7270	....	....	1290	....	....	5.7	6.8	6.8	6.8	16.8	...	...
		Minimum	6520	....	....	980	....	....	5.4	6.4	6.5	6.8	14.4	...	...
57	Gray Granite (?)	Average	6330	....	....	1140	....	....	4.2	7.8	7.9	8.6	18.6	50[a]	...
		Maximum	7070	....	....	1350	....	....	4.8	8.5	8.7	10.1	21.2	...	...
		Minimum	5470	....	....	1020	....	....	3.3	6.7	6.8	7.0	15.7	...	...
58	Light Gray Limestone	Average	1550	....	....	....	....	....	...	...	...	...	...	18	25
		Maximum	1640	....	....	....	....	....	...	...	...	...	...	20	...
		Minimum	1500	....	....	....	....	....	...	...	...	...	...	15	...

[b] 5 hour total absorption in boiling water after 48 hours in water at room temperature.

(a) Minimum thickness of facing given. In some specimens the thickness of facing increased at the edges so that it was possible to obtain full size specimens of facing material.

(b) Specimens containing both facing and backing were tested with the facing placed on the under side, so that the failure in tension occurred in the facing.

[a] Indicates that the number of cycles of treatment have been given, and if in column marked "first sign," no indication of failure was noticed, and if in column marked "end point" that the end point had not yet been reached.

The number in brackets indicates the number of specimens which reached the first sign or end point at the value noted.

[c] Where only average values are given, the maximum and minimum values are so close to the average that nothing is to be gained by giving them also.

Testing Concrete for Absorption

By Fred Weigel*

Since the use of concrete and concrete products in the building industry is increasing widely the question of absorption or more particularly permeability is becoming of vital importance. Permeability being the controlling factor with respect to the resistance of a material to the effects of fire, freezing, long periods of exposure to water and general weathering conditions, it is imperative that a test for absorption or permeability should be indicative of the resistance of this material to the conditions mentioned.

In connection with the manufacture of concrete trimstone it has been necessary for the writer to make numerous physical tests and study the reports of tests made by various commercial laboratories. A summary of several reports from commercial laboratories shows the following approximate results:

Compression, average conditions................	6000 to 7000 lb. per sq. in.
Compression after freezing in cold room.........	4000 to 4500 "
Compression after steaming....................	3800 to 4000 "
Absorption per cent of dry weight..............	7 to 8 per cent
Compression after absorption tests..............	4000 to 4500 lb. per sq. in.
Compression after fire test (heated at 700 deg. C.)	1500 to 2000 "

After studying the results of these tests the question arises as to the veracity of the exceptionally high figure for absorption as compared with the strength after freezing of 4000 to 4500 lb. per sq. in. or after the fire test of 1500 to 2000 lb. per sq. in. It seems highly improbable that a material carrying as much as 8 per cent by weight of free moisture could show a compressive strength of 4500 lb. per sq. in. after having been frozen at a temperature below 0 deg. F., or a strength of 2000 lb. per sq. in. after being heated to a temperature of 700 deg. C.

As a possible solution to this question it is suggested that some of the hydration products of the cement are being broken down during the continued heating and that some of the combined water as well as the free moisture that is held in the pores of the concrete is being driven from the cement paste. In this connection we quote F. R. McMillan of the Portland Cement Association: "It is the general thought at the present time that the hydration products of cement are both crystalline and amorphous materials, the amorphous material greatly predominating during the early stages with a growth of crystalline material taking place with increasing age of the concrete. It is probable that at no time do

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the hydration products become entirely crystalline and that the hydrated cement will always contain a sufficient quantity of the 'gel' to cause the cement paste as a whole to exhibit the properties of a colloidal gel. This gel is capable of assuming a variable water content which is fixed by the conditions of temperature and water vapor pressure of the atmosphere in which it is exposed. Under conditions of high temperature the gel tends to lose water; under conditions of low temperature or of high humidity, it tends to take on water." The tests described in the following paragraphs seem to bear out these conclusions quite closely.

Samples were taken from blocks of concrete stone which had been stacked in the yard and exposed to the weather for about one year. The samples were soaked in water at room temperature until they showed

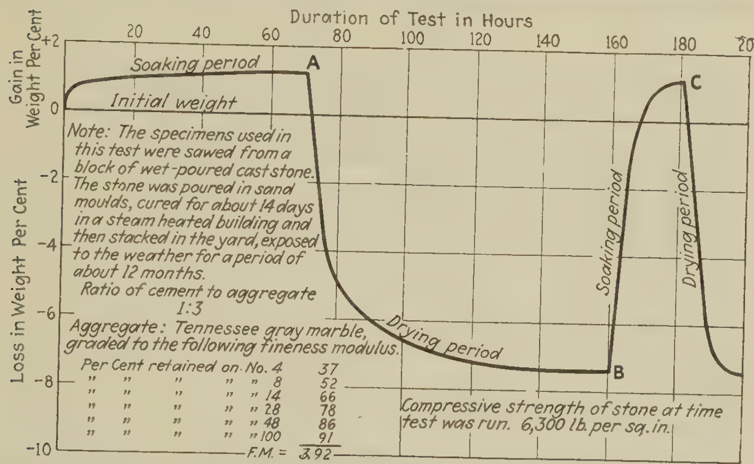

FIG. 1—VARIATION IN WEIGHT OF CONCRETE ALTERNATELY SOAKED IN WATER AT 70 DEG. F. AND DRIED IN AN ELECTRIC OVEN AT 110 TO 130 DEG. C.

no more gain in weight. This required from 72 to 96 hr. Upon breaking the specimens water was found to have penetrated to a depth of less than ½ in. and the centers of the specimens were from all appearances perfectly dry. Pieces chipped off could be beaten into a perfectly dry powder, and gave no indication of containing an appreciable amount of free moisture. The gain in weight during this period of soaking was in most cases from 1 to 1½ per cent and in no case did it exceed 2 per cent. However, these same specimens when placed in an electric oven and dried to a constant weight at a temperature of from 110 to 120 deg. C showed a loss in weight of from 8 to 10 per cent.

To illustrate this point a curve was plotted (Fig. 1) showing the rate of change in weight of specimens of the stone mentioned above during alternate periods of soaking and drying. During the first period of soaking, 72 hr. was required to bring the specimens to a constant

weight. At the point "A" on the curve, the specimens were placed in the oven at a temperature of from 110 to 130 deg. C and allowed to remain until they showed no more loss in weight. This required about 70 hr. After the curve passes the point "B", there is considerable indication that the physical properties of the concrete have been changed. The second period of soaking required only 20 hr. and the second period of drying was shortened in proportion. It is evident that during the first period of drying considerably more heat was required to reduce the specimens to the dry weight than in the second drying period. This gives us sufficient reason to believe that during the first drying period the mere evaporation of water was not the only action taking place. Likewise, the rapid absorption of water during the second soaking period would indicate that the voids left by the breaking down of the products

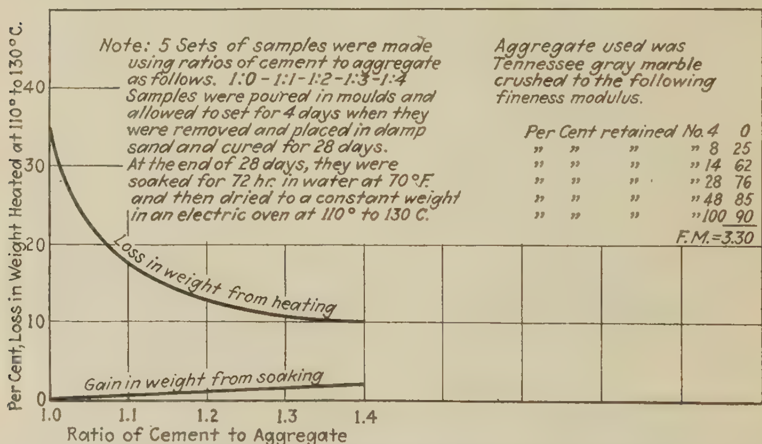

FIG. 2—EFFECT OF HEATING AT MODERATE TEMPERATURES ON CONCRETE HAVING VARIABLE RATIOS OF CEMENT TO AGGREGATE

of hydration during the first drying period were being filled with water.

Assuming the conclusions in the foregoing paragraphs to be correct, it would naturally follow that for varying proportions of cement to aggregate we would have a variable loss in weight due to heating. More specifically if all or a part of the water which has gone into combination with the cement is being driven out, the loss in weight when heated would vary directly with the amount of cement contained in the mix, all other factors remaining constant.

Following up this assumption the curve in Fig. 2 was plotted, using the loss in weight while drying at 110 to 130 deg. C for the vertical ordinates and the ratio of cement to aggregate for the horizontal ordinates. Five sets of specimens were made for this test, using the following ratios of cement to aggregate: 1:0 (neat cement), 1:1, 1:2, 1:3 and 1:4. The specimens were poured in tin moulds and allowed to set for 4 days, when

they were removed and placed in damp sand and cured for a period of 28 days. At this time they were soaked in water at 70 deg. F. for 72 hours and then dried to a constant weight in the oven at 110 to 130 deg. C. The loss in weight varied from 35 per cent of the dry weight for the neat cement to 10 per cent for the 1:4 mix. It will be noted that the slope of the curve diminishes as the ratio of the cement to aggregate decreases. This is explained by the fact that the gain in weight during the soaking period increases as the ratio of cement to aggregate decreases, which is to be expected. It is quite probable that if the curve had been extended to a ratio of cement to aggregate of 1:8 or 1:10 giving a very poor concrete, the absorption of water during the soaking period would have been great enough to entirely offset the decreasing loss in weight due to the low cement to aggregate ratio and the curve would assume an upward direction.

It might be interesting to note that all of the specimens used in the test just described retained an amount of water equal to 12½ per cent of the weight of the cement, which could not be driven out after continued heating at a temperature of 140 to 150 deg. C. This is consistent with Mr. McMillan's statement that the hydration products of cement are both crystalline and amorphous materials. The crystalline materials having a fixed water content while the amorphous materials are capable of assuming a variable water content which is fixed by the conditions of temperature and humidity of the atmosphere.

The specimens used in plotting curve No. 2 after having been thoroughly dried out were stacked on a shelf in a dry room, and their weights checked regularly over a period of four months. The cubes of neat cement increased about 15 per cent by weight of the cement in this time and the other mixes in proportion. This indicates that the hydration products broken down during the drying period were gradually taking on moisture from the air and that the concrete was assuming its original internal structure. The specimens at the end of the drying period had a white chalky calcined appearance, but at the end of the 4-months period of observation, they had taken on the hard glazy crystalline appearance, common to well-cured concrete; this crystalline structure extended to a depth of ¼ to ½ in. This was particularly noticeable in the cubes of neat cement. Shrinkage cracks and numerous hair lines had almost disappeared.

A New Permeability Test—The test for absorption recommended by the Bureau of Standards involves the drying of the specimens to a constant weight at a temperature exceeding 110 deg. C and a subsequent boiling in water for a period of 5 hr. The absorption is the difference between the dry weight and the weight after boiling expressed in per cent of the dry weight. In view of the findings already discussed we believe that this test is of value only when testing concretes having the same ratio of cement to aggregate and is certainly unfair when used as a basis for comparing concrete products with natural materials such as brick, stone and tile.

In searching for a method of testing for permeability which would be more indicative of the resistance of the material to weathering conditions the device shown in Fig. 3 was developed. It consists of a ¾ in. to 3 in. pipe bushing bolted between two ¼ in. plates. A hole of a diameter equal to the inside diameter of the bushing is bored in the bottom plate and the test specimen 1 in. thick and 4 in. square, properly protected with rubber gaskets, is slipped between the bushing and the bottom plate. The face next to the bushing must be flat and smooth and the specimen must be fairly uniform in thickness. However, a

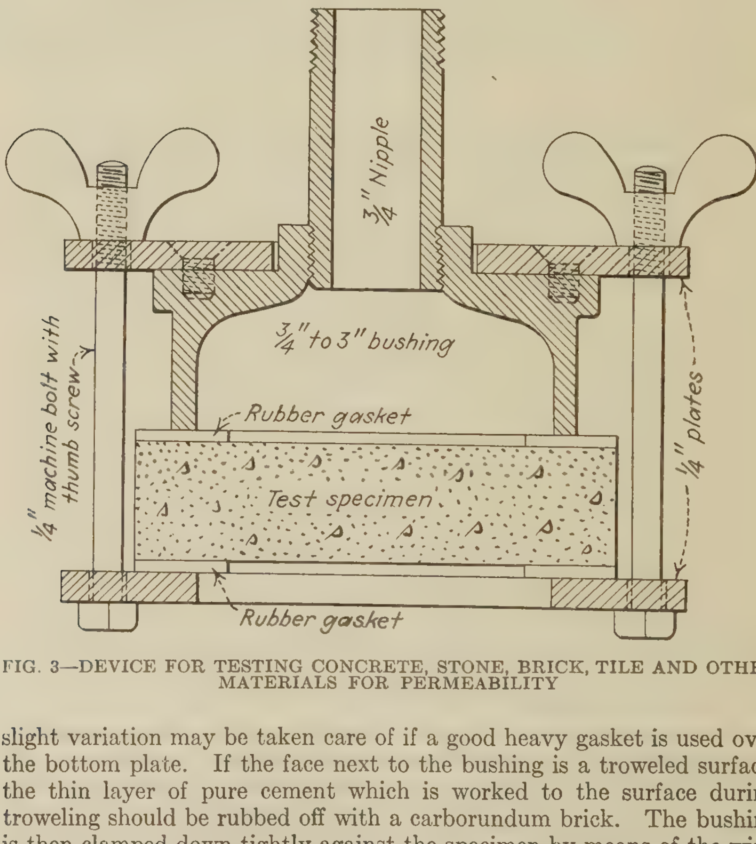

FIG. 3—DEVICE FOR TESTING CONCRETE, STONE, BRICK, TILE AND OTHER MATERIALS FOR PERMEABILITY

slight variation may be taken care of if a good heavy gasket is used over the bottom plate. If the face next to the bushing is a troweled surface, the thin layer of pure cement which is worked to the surface during troweling should be rubbed off with a carborundum brick. The bushing is then clamped down tightly against the specimen by means of the wing nuts at the top and water under 80 lb. pressure is admitted through the ¾ in. nipple. If leakage occurs between the bushing and the test specimen it may be eliminated by coating the gasket with red lead. The rate of penetration may be expressed as the time required for moisture to show on the outside of the 1 in. slab, or the specimen may be removed

and broken after one or two hours and the rate expressed in fractions of an inch per hour. The latter method is probably more satisfactory owing to the fact that in some cases the penetration is so slow that even after a certain amount of moisture is finding its way through the slab the evaporation may be rapid enough to keep the outside surface comparatively dry.

A comparison between the relative value of the method of testing commonly used and the test just described may be seen in the following table giving results on two parallel tests on specimens of a natural limestone and a wet poured concrete stone:

	Natural Stone	Concrete Stone
Compression	4100 lb. per sq. in.	6500 lb. per sq. in.
Absorption as indicated by prescribed method	5.5 per cent	7.1 per cent
Freezing with Sodium Sulphate—spalded	25 "	1 "
Compression after heating to 700 deg. C	disintegrated	1650 lb. per sq. in.
Penetration under 80 lb. water pressure	water spewed through 1-in. slab in one minute	$\frac{1}{8}$ in. per hour

No tests were made for steaming or freezing in cold room.

It may readily be seen that the figures shown for absorption give no indication whatever of the resistance of the materials to the conditions imposed in the remaining tests, while the figures for penetration are directly in keeping with the results obtained in the freezing and fire tests. However, if the specimens of concrete tested for freezing had been thoroughly dried out at 120 deg. C they would have been badly spalded, bearing out the conclusion that continued heating at this temperature breaks down the internal structure of the concrete leaving voids into which water rapidly penetrates. Water at 80 lb. pressure went through a specimen 1 in. thick which had been dried at 120 deg. C in 1½ hr., while it normally requires about 8 hr. to penetrate a 1 in. specimen in its natural condition. The compressive strength was reduced about 40 per cent by heating for 72 hr. at 120 deg. C.

If the moisture present in concrete in colloidal form is not transient during normal weathering conditions its presence with regards to impermeability, compressive strength, and resistance to freezing and fire should not be objectionable. Careful observation, over a period of four months, of specimens exposed to the weather showed only slight changes in weight. The loss in weight over a long period of hot dry weather was hardly enough to be detected. In most cases the weight of the specimens at the end of the period of observation showed a slight gain over the initial weight.

We realize that the device used in these experiments in testing for penetration is not likely to prove practical as a method of testing concrete in general. It is probably too severe a test for the average concrete

placed in construction jobs and very few of the concrete masonry units now on the market would pass it. Also there are a great many cast stone manufacturers who tamp the mix into rigid moulds in a dry or semi-dry condition. Owing to the fact that it is almost impossible to tamp the material in the moulds hard enough to close all the voids, a porous stone results. In order to hold the absorption down in a stone of this character, a waterproofing which reduces the capillarity of the pores in the stone is resorted to. Stone treated in this manner sheds water very well but offers practically no resistance to water under pressure. However, this device served admirably in demonstrating the questions brought up in this paper and that is the purpose for which it was primarily intended.

Discussion—Testing Concrete for Absorption

R. Van Niekerk (*Bombay, India, By Letter*).—I am in agreement with the findings contained in the above paper. The present method of testing for absorption is not only extremely unfair, particularly to concrete pipe, but appears to be entirely pointless. Mr. Van Niekerk.

Pipes stacked in the open may be dried at a temperature of somewhere near 110 deg. F., but are hardly likely to be boiled at any period of their existence in service. Moreover, as Mr. Weigel shows, the richer the mix and therefore, other things being equal, the higher the quality of the product, the poorer the results. Mr. Weigel's experiments also explain why pipes taken directly from the curing tanks will withstand high pressure while the same pipes after exposure to the sun for long periods will sometimes "sweat" under pressure. It is more or less generally recognized that "sweating" will disappear in a day or two and that specimens broken from such pipes show little or no penetration.

In the case of centrifugally cast pipes the unfairness of the present test is more marked. The process produces a neat cement finish on the interior surface which is highly desirable from every point of view but which would cause specimens of this type of pipe to show at a disadvantage compared with poured pipe under the present method of testing. It appears therefore that the only fair method of testing concrete pipe for absorption is an internal hydraulic test at normal temperature.

It may be contended that for centrifugal pipe an external pressure test would also be desirable owing to the different nature of the inner and outer surfaces. There would be no objection to this and Mr. Weigel's method would probably be difficult to improve on.

The Limitations of the Absorption Test for Concrete Products

By Raymond Wilson*

The use of the absorption test for concrete products is based on the natural assumptions that the absorption is a measure of the porosity of the concrete and that the porosity of concrete is a property closely related to its strength, water-tightness and resistance to weathering action.

The absorption of concrete or mortar is defined as the gain in weight of a dried sample after immersion in water, expressed as percent of the weight of the dry sample; it is approximately equal to the loss in weight when the concrete is dried from the saturated condition. The volume of the absorbed water expressed as percent of the volume of the sample is considered as the porosity of the concrete or mortar.

Concrete or mortar placed in a workable condition and allowed to harden without loss of water contains, in its saturated condition, a total quantity of water substantially equal to the quantity used in mixing. When saturated concrete is dried the loss in weight is this quantity of water less that portion of it which has gone into such a state of combination with the cement that it will not be evaporated by the drying conditions imposed.

The term "water of crystallization," sometimes applied to the chemically combined water in cement paste, has been avoided in this paper as it is by no means certain that the products of hydration are to any considerable degree crystalline. The bulk of the hydration products of portland cement appear to be amorphous materials containing water in varying degrees of combination with the cement. The proportions of water and cement in the mixture and the duration and temperature of curing influence the quantity of water retained by hydrated cement after drying to constant weight at any specified temperature. The temperature at which hydrated cement is dried likewise affects the quantity of water retained in the dried paste. Any differentiation between "free water" and "combined water" predicated on drying the hydrated cement under a specified condition must, so far as we now know, be based on an arbitrary choice of temperature and an arbitrary definition of what constitutes combined water. If it were not for this indefiniteness regarding the quantity of water remaining as a part of dried hydrated cement

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it would be possible to calculate the loss in weight when a saturated concrete of known composition is dried. A somewhat more extended discussion of these points will be found in a later portion of this paper.

The generally accepted method of bringing concrete to the dry condition as a preliminary to the absorption test is to heat it at a temperature somewhat above that of boiling water until it ceases to lose weight.* This method of drying has aroused some adverse criticism on the ground that it causes the concrete to lose more water than it would lose on exposure to any usual atmospheric condition. This places concrete at a disadvantage when compared by means of the absorption test with other materials, the degree of drying out of which is less affected by the drying temperature. As an alternative, it has been variously suggested that drying be accomplished by heating at a lower temperature, either until the weight is constant, or for a definite period of time.

The data and discussions presented in this paper show that neither of the assumptions on which the absorption test is based is applicable to concrete, and that, as a direct measure of the quality of cast stone or other concrete products, the present absorption tests are of doubtful value. In the development of this theme the following factors have been considered:

(1) Quality of the concrete as affected by water-cement ratio and curing.
(2) Loss in weight of concrete on drying.
(3) Absorption of water by dried concrete.
(4) Rates of loss in weight and absorption.
(5) Temperature employed in drying concrete as a preliminary to the absorption test.
(6) Size and shape of the specimen.

METHODS OF TEST

The tests reported here were made in two series. In the earlier series, identified in this paper as mortar tests, absorptions were determined on a series of mortars covering two periods of curing and a wide range of proportions of cement to aggregate. Tests designed to show the effect of the absorption test procedure on the compressive strengths were also included. The later series, identified as concrete tests, was designed to show the effect of size and shape of test specimen on (1) the loss in weight on drying, (2) the absorption, and (3) the rates of loss and absorption. Other tests in this series show the effect on the compressive strength of drying and of drying followed by immersion for various periods.

The specimens in both series were dried at each of two temperatures, 167 deg. F. and 239 deg. F. (75 deg. C. and 115 deg. C.) in an electrically heated and controlled oven. Absorption tests were made by immersing

* *Proceedings*, American Concrete Institute, 1925, p. 603.
Standards, American Society for Testing Materials, 1927, pp. 185, 197.

the dried specimens in water at room temperature; after various immersion periods they were taken from the water and weighed, after removing the excess water with a damp cloth.

Mortar Tests—The specimens were 2-in. cubes, molded 6 in a batch, 3 of which were used for strength tests in the usual manner and 3 for drying and absorption followed by compression tests. All specimens were cured one day in the forms and thereafter stored in the moist room. Strength specimens were cured in the moist room until test and tested wet. The absorption specimens were cured in the moist room for periods two days less than the corresponding strength specimens. Duplicate sets were then dried to constant weight at the two temperatures employed and immersed in water for two days, the gain in weight at various periods being noted. After this immersion, the specimens were tested for compressive strength, in the wet condition.

The total period of wet curing of both the ordinary strength specimens and those which had been subjected to absorption was thus the same, although the absorption specimens were actually two to five days older than the others when tested, due to the time required for drying to constant weight. This equalization of the total moist curing period of the two sets of specimens was, of course, based on the well-founded belief that strength increases take place in concrete only while uncombined water is available for combination with the cement. Another factor, the effect of which cannot be closely evaluated, is that there may be a temporary increase in the rate of hydration of the cement during the earlier part of the drying when the temperature has been moderately increased and before the water still available for combination with the cement has been evaporated.

Three mortar mixes were tested: 1:2, 1:4, and 1:6. The consistency of all mortars was approximately the same—that which would give a flow of about 160 on a mortar flow table. The sand was a calcareous aggregate from Elgin, Ill., graded 0–No. 4. The cement was a mixture of equal parts of four brands of portland cement purchased in the Chicago market. The composition of the batches used in the various mortars is given in Table 1.

Water Retained by Hardened Cement Paste—It was inferred in an earlier part of this paper that the loss in weight, when a saturated concrete was dried until no further loss occurred, could be calculated if the proportions of the mixture and the quantity of water in a state of stable combination with the cement were known. It seemed desirable to check this belief, determining the quantity of water so combined with the cement by an accurate technique, and using the values found to calculate the loss in weight. Accordingly determinations of the water contained in the dried hydrated cement were made as follows: The cement was mixed with water in the proportions used in the three different mortars, and a sufficient quantity of a finely powdered inert material added to the mixture to give a consistency such that there was no separation of water after placing. After about two days of hardening in a tightly

stoppered tube the paste was ground to a powder and kept in a sealed container until tested. Seven days and twenty-eight days after they were prepared, portions of each of these pastes were dried to constant weight at 167 deg. F. and 239 deg. F., and the quantity of water retained by the cement under these conditions determined by igniting the dried sample to constant weight at about 1800 deg. F., at which temperature the water retained after drying at the lower temperature was completely driven off.

These pastes inadvertently received two days longer moist curing than were received by the corresponding mortars previous to the beginning of the drying treatment. It is believed that this difference in curing time did not seriously affect the comparison of calculated losses with observed losses.

The rate and extent of hydration of the cement may not be exactly the same in this type of mixture as in an ordinary mortar or concrete but any differences are probably slight. Tests have shown that grinding the paste at an early age has no detectable effect on the rate of hydration provided the paste is protected from evaporation during the grinding and subsequent storage. Another factor which is probably of greater importance is the difference in the time required to dry the small sample of paste (about 1 gr.) and the 2-in. mortar cube. While the sample of paste is quickly heated to the temperature of the oven and is dried completely in a few hours the mortar is heated and dried more slowly and thus has a longer time during which hydration may progress at a temporarily increased rate. This would tend to lessen any discrepancy caused by the difference, in these tests, in the period of moist curing of the pastes and the mortars.

Concrete Tests.—The aggregate used in this series was from Elgin, Ill., graded up to ¾-in. The proportions of cement and aggregate were so chosen that the concrete had a slump of 3 to 4 in. with water-cement ratios of 5.0 and 7.5 gal. per sack of cement.

In the tests designed to show the effect of size and shape of specimen on the rates of loss in weight on drying and of absorption, each of these concretes was molded into 6 x 6-in. and 3 x 6-in. cylinders, 6 x 4-in., 6 x 2-in., and 6 x 1-in. disks, and 4-in. and 2-in. cubes. The specimens were cured in water until 14 days old, after which they were dried to constant weight at the same temperatures as in the mortar tests. In the absorption tests, which were made only on the concrete dried at 239 deg. F., the weights of the specimens (after surface drying) were noted after immersion for ½, 3, 6, 24, 48, 72, 96 and 144 hrs., and after a 5-hr. boiling which immediately followed the 144-hr. immersion.

For the strength tests 2 sets of eighteen 2-in. cubes of each of the mixtures used in the tests just described were cured in water until 14 days old. Two cubes from each set were tested in compression and the remainder dried either at 167 deg. F. for 4 days or at 239 deg. F. for 2 days. Two cubes of each set were tested dry; the remaining 14 cubes were immersed in water and two withdrawn and tested after each of the

immersion periods: $\frac{1}{2}$, 3, 6, 24, 48, 72, and 96 hr. The tests were repeated one week later. Absorption tests made on these specimens were in agreement with the other results and are not presented here.

Discussion of Data on Drying and Absorption

The essential data obtained in this investigation are shown in Tables 1 to 11.

Effect of Mix.—The loss in weight on drying and the absorption of the dried mortar cubes, when immersed in water at room temperature, are shown in Table 2. Both the loss in weight and the absorption are higher in the lean than in the rich mixes, though the differences are not great, in fact not much larger than the differences which may occur between supposedly duplicate samples of concrete. Such small differences in absorption obviously cannot be taken as indicative of the relative quality of 1:2, 1:4, and 1:6 mortars with 28-day compressive strengths of 6490, 2420 and 990 lb. per sq. in. respectively.

The explanation of this failure of the absorption test to indicate quality is suggested by the last two columns of Table 1, where the water content is expressed first in terms of the cement and second in terms of

Table 1—Composition of Mortar Batches

Mix by Volume	Cement, grams	Sand, grams	Water		
			cc.	Per Cent by Weight of Cement	Per Cent by Weight of Cement plus Sand
1:2	800	1905	350	44	12.9
1:4	400	1905	310	78	13.4
1:6	275	1960	310	113	13.9

the total quantity of dry materials. The range of water content of the three mortars, in terms of the dry materials, is only from 12.9 to 13.9 per cent, while in terms of the cement the range is from 44 to 113 per cent. The absorption of the mortar will naturally parallel the total quantity of water in the mixture while strength and other useful properties are recognized as being, in large measure, dependent on the ratio of water to cement, that is, on the composition of the cement paste. This same small variation in total water content is true of any series of mortars or concretes in which the proportion of cement to aggregate is varied, while the grading of the aggregate and the workability, as far as possible, are kept constant. That is, for these conditions the ratio of water to dry materials has very nearly a constant value.

It is only as changes in the grading of the aggregate, or in the consistency (in the sense of wetness or dryness) of the mix are introduced, that larger variations are found in the quantity of mixing water in its

relation to the dry materials. Because of the difference in the quantity of paste present it is possible for a lean concrete or mortar mix of high water-cement ratio, in which a well-graded aggregate is used, to have a lower absorption than a rich one of low water-cement ratio, where the aggregate is poorly graded.

It is this measurement of the porosity of the dried concrete in terms of the whole concrete volume, rather than in terms of the cement paste, that constitutes one of the two inherent weaknesses of the absorption

TABLE 2—DRYING AND ABSORPTION TESTS OF MORTARS

Mix by volume; consistency to give flow of about 160 on 18-in. flow table.
Laboratory portland cement, a mixture of equal parts of 4 brands.
Tests made on 2-in. cubes dried in electrically heated and controlled oven.
Absorption test made by immersion in water at room temperature.
Absorption and loss in weight given as percent of weight after drying.
Cubes moist cured for 5 days (marked 7d.) or 26 days (marked 28d.) before drying.
Each value is average of results of 6 tests.

Period of Drying or Immersion.	Drying Temperature 167 deg. F.						Drying Temperature 239 deg. F.					
	1:2 Mix		1:4 Mix		1:6 Mix		1:2 Mix		1:4 Mix		1:6 Mix	
	7 days	28 days	7 days	28 days	7 days	28 days	7 days	28 days	7 days	28 days	7 days	28 days
LOSS IN WEIGHT ON DRYING												
3 hr.	2.85	1.93	5.07	2.74	6.49	3.95	5.13	4.47	7.87	6.27	9.68	7.93
7 hr.	4.24	3.23	6.99	5.14	8.97	6.72	7.25	6.13	9.67	8.69	10.41	10.80
24 hr.	6.84	5.54	9.53	8.44	10.48	10.44	8.93	6.83	9.90	10.05	10.45	11.05
48 hr.	7.93	6.78	9.73	9.39	10.50	10.58	8.96	9.03	9.90	10.12	10.45	11.08
72 hr.	8.36	7.43	9.74	9.50	10.54	10.62	9.03	9.07	9.94	10.14	10.48	11.09
96 hr.	8.44	7.74	9.74	9.54	10.54	10.62	9.03	9.08	9.94	10.14	10.48	11.09
144 hr.	8.53	7.97	9.74	9.54	10.54	10.63	....	....	....	.....	.....	.....
ABSORPTION												
½ hr.	4.02	2.13	7.56	3.39	8.57	5.64	3.30	2.37	7.38	2.84	8.62	4.43
3 hr.	7.24	4.65	8.30	7.63	8.77	9.02	7.14	4.77	8.62	6.02	8.77	9.10
6 hr.	7.43	7.08	8.44	8.36	8.87	9.09	7.89	5.97	8.70	7.63	8.80	9.37
24 hr.	7.57	7.23	8.57	8.47	8.93	9.18	8.10	7.81	8.78	8.93	8.93	9.62
48 hr.	7.63	7.34	8.64	8.53	8.96	9.24	8.23	8.17	8.93	9.10	8.98	9.71

test as a direct gage of concrete quality. The other weakness is considered in the succeeding paragraphs.

Effect of Drying Temperature.—Reference has been made to the possibility of drying the concrete at a lower temperature as a step toward making the absorption test more nearly indicative of the porosity which the concrete is likely to develop in service. The extent to which this procedure would affect the values for absorption is shown by comparing the losses in weight and the absorptions resulting from drying mortar

completely at 239 deg. F. and at 167 deg. F. The data are summarized in Table 3. The maximum difference in loss in weight occasioned by this difference (72 deg. F.) in the temperature of drying is 1.11 per cent, and the average difference for the three mortars at two ages is 0.49 per cent. The corresponding differences in absorption are 0.83 per cent

TABLE 3—EFFECT OF DRYING TEMPERATURE ON LOSS IN WEIGHT AND ON ABSORPTION

Summary of data from Table 2

	Drying Temperature, deg. Fahr.	1:2 Mix		1:4 Mix		1:6 Mix		Average	Maximum	Minimum
		7 days	28 days	7 days	28 days	7 days	28 days			
Loss on drying........	167 deg. F.	8.53	7.97	9.74	9.54	10.54	10.63	....	....	....
	239 deg. F.	9.03	9.08	9.94	10.14	10.48	11.09	....	....	....
	difference	0.50	1.11	0.20	0.60	0.06	0.46	0.49	1.11	0.06
Absorption...........	167 deg. F.	7.63	7.34	8.64	8.53	8.96	9.24	....	....	....
	239 deg. F.	8.23	8.17	8.93	9.10	8.98	9.71	....	....	....
	difference	0.60	0.83	0.29	0.57	0.02	0.47	0.46	0.83	0.02

TABLE 4—RATES OF DRYING AND OF ABSORPTION

Values are percents of total loss or absorption based on data in Table 2.

Period of Drying or Immersion.	Drying Temperature 167 deg. F.						Drying Temperature 239 deg. F.					
	1:2 Mix		1:4 Mix		1:6 Mix		1:2 Mix		1:4 Mix		1:6 Mix	
	7 days	28 days	7 days	28 days	7 days	28 days	7 days	28 days	7 days	28 days	7 days	28 days
	LOSS IN WEIGHT ON DRYING											
3 hr.............	33	24	52	29	62	37	57	49	79	62	92	72
7 hr	50	41	72	54	85	63	80	68	97	86	99	97
24 hr............	80	70	98	88	99	98	99	75	100	99	100	100
48 hr............	93	85	100	98	99	99	90	100	100	100	100	100
72 hr............	98	94	100	99	100	100	100	100	100	100	100	100
96 hr............	99	97	100	100	100	100	100	100	100	100	100	100
144 hr...........	100	100	100	100	100	100	...	...	...	...	...	...
	ABSORPTION											
½ hr............	53	29	88	40	96	61	40	29	83	31	96	46
3 hr.............	95	63	96	89	98	98	88	58	97	66	98	94
6 hr.............	97	97	98	98	99	98	96	73	98	84	98	97
24 hr............	99	99	99	99	100	99	98	96	98	98	100	99
48 hr............	100	100	100	100	100	100	100	100	100	100	100	100

maximum and 0.46 per cent average. As might be expected, the difference is greatest in the rich mixes where the proportion of cement paste is the largest.

Rate of Loss in Weight and Absorption.—The rich mixes dry out and absorb water more slowly than lean ones, and mortar 28 days old

dries out and absorbs water more slowly than similar mortar 7 days old. This relationship is shown in Table 2, and brought out more clearly in Table 4, in which the losses and absorptions are shown as percentages of the total loss or absorption.

Drying is very much slower at 167 deg. F. than at 239 deg. F. The 28-day old 1–2 mortar required over 96 hr. to reach a weight within 0.2 per cent of its final weight when dried at 167 deg. F., while at 239 deg. F. it reached a comparable degree of dryness in 48 hr. The 7-day old 1:6 mortar was almost completely dry in 7 hr. at 239 deg. F. and in 24 hr. at 167 deg. F.

There is no regular difference in the rate of absorption of mortars dried at the two temperatures. There seems to be a general tendency, emphasized in some instances, for absorption to take place more rapidly in the mortars dried at the lower temperature. The results are hardly concordant enough to warrant conclusions or even speculations on this point.

Effect of Drying Concrete Under Normal Atmospheric Conditions.—There is nothing in these tests to show quantitatively the effect of drying concrete or mortar under the conditions to which cast stone would be subjected in service. Enough data are available from other sources to indicate in a general way what the results would be and to demonstrate that the absorption test is not a true measure of the porosity which concrete might develop in service.

Schlick* has made tests on concrete drain tile and on laboratory specimens in which he employed a number of drying temperatures covering a wider range than those used in the tests reported here. The average losses on drying, taken from Schlick's curve showing the relation of loss to drying temperature, are 7.4, 6.9, 5.7 and 4.3 per cent for concretes dried at 115, 75, 40 and 25 deg. C. (239, 167, 104 and 77 deg. F.) These results show that if completely dried at any normal atmospheric temperature the concrete would have an absorption less than that shown in any present standard absorption test.

Furthermore, it is extremely unlikely that concrete in any ordinary exposure would ever dry even to the extent indicated in Schlick's tests. The much lower rate of drying at the lower temperatures prevents complete drying out between additions of water by rainfall, condensation, or other agencies. When the differences in rate of drying and absorption occasioned by variations in the water-cement ratio and the period of curing are taken into account it is seen that results of porosity determinations on a group of miscellaneous concretes by any present standard absorption tests may not even parallel the actual porosities of the concretes in service.

This is the second inherent weakness of the absorption test as a measure of the quality of concrete products. The fundamental error lies in the assumption that all the water which may be evaporated from

* "Effect on Concrete of Immersion in Boiling Water and Oven Drying." W. J. Schlick, Bulletin 59, Engineering Experiment Station, Iowa State College (1921), Fig. 9, p. 19.

concrete is present simply as free water filling a series of pores the sizes and locations of which are definitely and permanently fixed. It is more nearly in accord with known facts to consider at least a part of this water, perhaps a considerable part of it, as being in a state of combination with the cement, the degree of which is fixed by external conditions. The water in this state of combination functions as an integral part of

Table 5—Water Retained in Dried Hydrated Cement

See text for details of test procedure.
Values are percent water retained in terms of unhydrated cement.

Water Used in Pastes, per cent by weight of cement	Water Retained in Combination after Drying			
	At 167 deg. F.		At 239 deg. F.	
	Cured 7 days	Cured 28 days	Cured 7 days	Cured 28 days
44	13.7	17.6	11.5	14.3
78	16.1	20.4	13.1	16.9
113	17.4	22.5	14.0	18.2

the cement paste, thus reducing the volume of voids filled with free water.

With extremely porous concrete it would, of course, be impossible to close the pores by this development of the hydrated cement paste. In concrete of fairly low water-cement ratio which has been adequately cured (in water-tight concrete, for instance) the volume of pore space

Table 6—Comparison of Observed Absorption and Loss in Weight on Drying With Calculated Loss

See text for method of calculating loss on drying.

	Drying Temperature, deg. F.	1:2 Mix		1:4 Mix		1:6 Mix	
		7 days	28 days	7 days	28 days	7 days	28 days
Observed loss on drying	167	8.53	7.97	9.74	9.54	10.54	10.63
Calculated loss on drying	167	8.60	7.34	10.38	9.60	11.47	10.78
Absorption	167	7.63	7.34	8.64	8.53	8.96	9.24
Observed loss on drying	239	9.03	9.08	9.94	10.14	10.48	11.09
Calculated loss on drying	239	9.22	8.37	10.95	10.22	11.92	11.32
Absorption	239	8.23	8.17	8.93	9.10	8.98	9.71

is probably small compared with the volume it reaches when the concrete is completely dried at temperatures used in the standard test, which removes water normally in a state of useful combination with the cement. It is the size and extent of the pores present in the concrete under normal conditions, rather than after complete drying at higher temperatures, which determine the extent of the destructive effect of weathering.

This picture of the structure of a hydrated cement paste assists

in a better understanding of some physical properties such as the resistance to flow of water by a material which after drying may have a fairly high proportion of pore space.

Comparison of Calculated and Observed Loss in Weight.—In Table 5 are shown the quantities of water retained in dried hydrated cement pastes, as determined by the special technique described above under

Table 7—Effect of Size of Specimen on Loss on Drying

Drying tests on concrete.
Concrete cured in water 14 days before drying.
Aggregate: sand and pebbles from Elgin, Ill., graded 0–⅜-in.
Cement· a laboratory mixture of 4 brands of portland cement.
Values for concrete dried at 239 deg. F. average of 2 tests made at different times; single tests only on concrete dried at 167 deg. F.

Drying Period, hr.	Loss in Weight on Drying—Per Cent of Dry Weight													
	6 x 6 in. Cylinder		3 x 6 in. Cylinder		6 x 4 in. Disk		6 x 2 in. Disk		6 x 1 in. Disk		4 in. Cube		2 in. Cube	
	5.0*	7.5*	5.0	7.5	5.0	7.5	5.0	7.5	5.0	7.5	5.0	7.5	5.0	7.5
Drying Temperature 167 deg. F.														
3	0.34	0.49	0.64	0.99	0.32	0.80	1.26	1 68	2.34	2.09	0.83	1.10	1.63	2.10
6	1.10	1.50	1.53	2.31	1.19	1.73	2.22	3.05	3.29	3.63	1.62	2.24	2.71	3.54
24	2.90	3.83	3.50	5.09	3.09	4.00	3.96	5.37	5.44	6.45	3.41	4.37	4.69	5.84
48	3.71	4.91	4.33	6.15	3.93	5.23	4.68	6.27	6.32	7.00	4.25	5.34	5.43	6.65
72	4.13	5.47	4.84	6.68	4.34	5.77	5.13	6.65	6.52	7.07	4.67	5.81	5.69	6.71
120	4.80	6.06	5.41	7.08	5.17	6.51	5.84	6.94	6.65	7.11	5.33	6.38	6.05	6.77
144	4.89	6.29	5.67	7.14	5.33	6.75	5.99	7.00	6.69	7.15	5.45	6.53	6.10	6.80
168	5.15	6.53	5.80	7.14	5.56	6.91	6.08	7.05	6.70	7.15	5.63	6.61	6.10	6.80
192	5.30	6.66	5.98	7.14	5.66	6.97	6.13	7.10	6.71	7.17	5.75	6.70	6.12	6.80
216	5.39	6.78	5.98	7.14	5.83	7.03	6.23	7.10	6.71	7.17	5.83	6.70	6.12	6.80
240	5.56	6.84	....	....	6.06	7.06	6.28	....	....	....	5.92	....	....	....
288	5.70	7.02	....	....	6.06	7.16	6.28	....	....	....	6.00	....	....	....
312	5.73	7.02	....	....	....	7.16	....	....	....	....	6.00	....	....	....
Drying Temperature 239 deg. F.														
3	0.96	1.13	1.29	1.73	1.14	1.07	1.85	2.75	2.49	3.78	1.49	1.49	2.63	3.26
6	2.00	2.57	2.21	3.03	1.97	2.65	2.73	4.09	3.61	5.17	2.50	3.12	3.64	4.50
24	4.39	6.39	5.23	6.67	5.09	5.39	4.87	6.77	6.33	7.28	4.60	5.89	5.92	6.67
48	6.18	7.34	6.32	7.13	6.15	6.98	6.31	7.42	6.54	7.37	6.03	6.95	6.24	6.85
72	6.42	7.37	6.37	7.18	6.45	7.09	6.41	7.46	6.59	7.43	6.22	6.97	6.31	6.88
120	6.57	7.38	6.43	7.19	6.64	7.17	6.47	7.47	6.63	7.46	6.30	7.04	6.34	6.90
144	6.57	7.39	6.54	7.26	6.66	7.20	6.55	7.48	6.65	7.47	6.36	7.08	6.36	6.91

* Gallons water per sack of cement.

"Methods of Test." From these values and the quantities of cement, aggregate, and water in the mortars, the losses in weight expected of the mortars were calculated. These calculated losses are shown in Table 6 which also gives the loss on drying and the absorption observed in the tests on the mortars. The agreement is as good in most cases as that generally found in absorption tests. These tests are only of

incidental importance except as they throw light on one of the factors affecting the absorption test.

Relation of Loss on Drying to Absorption.—It will be noted in Table 3 that in all cases the gain in weight on absorption is less than the loss on drying. The average ratio of the absorption to the observed loss in weight is 0.89. In planning these tests it was assumed that a 48-hr. period of immersion would be sufficient to saturate the mortars thoroughly.

TABLE 8—EFFECT OF SIZE OF SPECIMEN ON RATE OF LOSS ON DRYING

Values are loss to the end of each period expressed as percent of total loss. Data from Table 7.

Drying Period, hr.	6 x 6 in. Cylinder		3 x 6 in. Cylinder		6 x 4 in. Disk		6 x 2 in. Disk		6 x 1 in. Disk		4 in. Cube		2 in. Cube	
	5.0*	7.5*	5.0	7.5	5.0	7.5	5.0	7.5	5.0	7.5	5.0	7.5	5.0	7.5
DRYING TEMPERATURE 167 DEG. F.														
3	6	7	11	14	5	11	20	24	35	29	14	16	27	31
6	19	21	25	32	20	24	35	43	49	51	27	33	44	52
24	51	55	59	71	51	56	63	76	81	90	57	65	77	86
48	65	70	72	86	63	73	75	88	94	98	71	80	89	98
72	72	78	81	94	72	81	82	94	97	99	78	87	93	99
120	84	87	90	99	85	91	93	98	99	99	89	95	99	100
144	85	90	95	100	88	94	95	99	100	100	91	98	100	100
168	90	93	97	100	92	96	97	99	100	100	94	99	100	100
192	92	95	100	100	93	97	98	100	100	100	96	100	100	100
216	94	97	100	100	96	98	99	100	100	100	97	100	100	100
240	97	97	...	...	100	99	100	...	...	...	99	...	...	...
288	99	100	...	...	100	100	100	...	...	...	100	...	...	...
312	100	100	...	...	...	100	...	...	...	...	100	...	...	...
DRYING TEMPERATURE 239 DEG. F.														
3	15	15	20	24	17	15	28	37	37	51	23	21	41	47
6	30	35	34	42	30	37	42	55	54	69	39	44	57	65
24	67	87	80	92	77	75	74	91	95	97	72	83	93	97
48	94	99	96	98	93	97	96	99	98	99	95	98	98	99
72	98	100	97	99	97	99	98	100	99	99	98	98	99	100
120	100	100	98	99	100	100	99	100	100	100	99	99	100	100
144	100	100	100	100	100	100	100	100	100	100	100	100	100	100

* Gallons water per sack of cement.

In order to make the compression tests after the same period of moist curing as that of the cubes which had not been subjected to drying, the absorption test was terminated at 48 hr., even though the cubes had not reached a constant weight in most cases at that time. However, other work has indicated that little further absorption would have taken place if the immersion had been continued beyond the 48-hr. period.

Schlick* reports that absorption tests of concrete drain tile by the 5-hr. boiling test give results averaging about 30 to 40 per cent higher than those obtained by immersion at room temperature for 72 hr., and

* "Effects on Concrete of Immersion in Boiling Water and Oven Drying." W. J. Schlick Bull. 59, Engineering Experiment. Station, Iowa State College (1921), Table 3, p. 17, and Table 4, p. 18.

that the loss on drying is approximately midway between the results of the absorption tests by the two methods.

Effect of Size of Specimen.—Data on the effect of size of test specimen on the loss in drying at the two temperatures are given in Table 7. These tests were made on the two concrete mixtures. The rates of loss at the two temperatures are shown in Table 8, in which the losses after different periods of drying are given as percentages of the total loss. While the total loss for the different sized specimens is in good agreement there is a marked difference in the rate of drying in the earlier hours.

TABLE 9—EFFECT OF SIZE OF SPECIMEN ON AMOUNT AND RATE OF ABSORPTION

Absorption tests on concrete dried at 239 deg. F.
See Table 7 for drying and other data on concrete.
Each value is average of results of 2 tests made on different days.
Values in second part of table are absorption expressed as percent of loss on drying (Table 7).

Immersion Period, hr.	6 x 6 in. Cylinder		3 x 6 in. Cylinder		6 x 4 in. Disk		6 x 2 in. Disk		6 x 1 in. Disk		4 in. Cube		2 in. Cube	
	5.0*	7.5*	5.0	7.5	5.0	7.5	5.0	7.5	5.0	7.5	5.0	7.5	5.0	7.5
ABSORPTION EXPRESSED AS PER CENT OF DRY WEIGHT														
½	1.62	2.35	2.47	3.20	2.00	2.61	2.37	3.31	3.16	4.36	2.07	2.33	3.49	4.00
3	3.66	4.89	4.95	6.07	4.08	5.33	4.88	6.35	5.59	6.29	4.26	4.88	5.45	5.97
6	4.58	5.83	5.60	6.28	4.95	6.18	5.65	6.49	5.70	6.34	5.06	6.03	5.54	6.05
24	5.88	6.42	5.77	6.45	5.76	6.45	5.79	6.62	5.92	6.51	5.54	6.23	5.70	6.21
48	5.83	6.47	5.81	6.49	5.85	6.51	5.84	6.65	5.99	6.56	5.61	6.28	5.72	6.23
72	5.89	6.54	5.90	6.49	5.95	6.59	5.93	6.72	6.01	6.59	5.64	6.31	5.73	6.25
96	5.90	6.57	5.93	6.55	5.98	6.63	5.96	6.72	6.02	6.63	5.64	6.31	5.79	6.34
144	5.98	6.60	5.99	6.64	5.98	6.63	6.01	6.77	6.14	6.69	5.74	6.36	5.81	6.46
5 boiling	5.98	6.75	6.03	6.90	5.99	6.86	6.03	7.17	6.19	7.11	5.74	6.65	5.89	6.62
ABSORPTION EXPRESSED AS PER CENT OF LOSS ON DRYING														
½	25	32	38	44	30	36	36	44	48	58	33	33	55	58
3	56	66	74	83	62	74	75	85	84	84	67	69	86	86
6	70	79	86	86	74	86	86	87	86	85	80	85	87	87
24	90	87	88	89	86	90	88	88	89	87	87	88	90	90
48	89	88	89	89	88	90	89	89	90	88	88	89	90	90
72	90	89	90	89	89	92	91	90	91	88	89	89	90	91
96	90	89	91	90	90	92	91	90	91	89	89	89	91	92
144	91	89	92	91	90	92	92	91	92	89	90	90	91	93
5 boiling	91	91	92	95	90	95	92	96	93	95	90	94	92	96

* Gallons water per sack of cement.

This is of importance in considering the advisability of drying for a fixed time such as 24 hr. The effect that such a method of drying might have on the results of the absorption test, unless the size of the specimens were specified, is indicated by the results of the drying test as presented in Table 8. For example when the concrete made with 5 gal. water per sack of cement was dried at 239 deg. F., 0.94 of the total ioss from the 2-in. cube took place in 24 hr., while only 0.68 of the total loss from the 6 x 6-in. cylinder occurred in the same time, both specimens having nearly the same loss after 144 hr. or even after 48 hr. drying. At

the lower temperature, 167 deg. F., the 24-hr. losses for the 2-in. cube and the 6 x 6-in. cylinder were respectively 0.51 and 0.77 of the total losses.

The absorptions of the specimens after drying at 239 deg. F. are given in Table 9. The rates of absorption, as shown in the second part of Table 9, vary rather widely in the earlier periods among specimens of different sizes. The absorptions after 24 hr. immersion differ in most cases by less than 0.3 per cent from the final values (144 hr.).

Additional absorptions by the concrete of lower water-cement ratio during the 5-hr. boiling were negligible, but were as high as 0.4 per cent in some of the specimens of concretes of higher water-cement ratio.

In other respects these tests show the same type of results as the mortar tests. The difference in absorption caused by the difference in the two concrete mixes was about 0.7 per cent. The difference in loss on drying at the two temperatures was about 0.5 per cent. For specimens of the same size the rates of drying and absorption were greater in the concrete of higher water-ratio. The average ratio of gain on absorption to loss on drying was 0.90 in the concrete tests.

A Suggested Modification of the Absorption Test.—It has been shown in both the mortar tests and the concrete tests that the absorptions of concretes of widely varying quality may differ so little as to make the test of doubtful value as a criterion of quality when used alone. On the other hand, the quality of the concrete is reflected in the rate at which the absorption takes place, increased curing periods or decreased water-cement ratios having the effect of causing absorption to take place more slowly during the early periods of immersion. While the data presented here are not sufficient to fix the limiting values for a specification based on the *rate of absorption*, they strongly suggest that a specification of that form would be a more useful one than any present absorption specification for concrete products. The requirement under such a specification would take the form of fixing the maximum absorption at certain periods, say at the 1, 3, and 6-hr. period, in terms of the 24-hr. absorption. A limit on the 24-hr. absorption would serve to rule out extremely sloppy mixtures or mixtures in which the aggregate is so poorly graded as to require excessive mixing water and which are made rich in cement to meet strength requirements. That is, in effect, all that the present absorption specifications accomplish.

This type of specification when coupled with a direct strength requirement would more nearly represent a test of what is expected of the material in service and would differentiate on a basis of quality among various concretes. Such a test could make use of the advantages of the present standard method of drying without involving its disadvantages.

The apparent necessity of fixing the size of specimen would be an objection to this form of specification. Other objections not suggested by these tests might develop from a more extended investigation of its possibilities. Further work would also have to be done to test the applicability of this form of specification to dry-tamped products, as the tests

discussed in this paper were made on plastic mortars and concretes. It will also be necessary to give consideration to the effect which the absorption of the aggregates would have on such a method of test.

DISCUSSION OF STRENGTH DATA

Reference has been made in a previous paragraph to the fact there was no relation between the values shown by the absorption tests and the quality of concrete as indicated by its strength. It was pointed out that the absorptions of the three mortars 1:2, 1:4, and 1:6 showed practically the same value, whereas the 28-day strengths were respectively 6490, 2420, and 990 lb. per sq. in. Similar results were shown for the concretes tested.

The strength tests are of interest also in respect to the effect which

TABLE 10—COMBINED EFFECT OF DRYING AND 48-HR. IMMERSION ON STRENGTH OF MORTARS

Compression tests of 2-in. cubes.
All cubes tested wet; 7 and 28-day cubes for drying and absorption tests removed from moist room at 5 and 26 days, dried, then immersed for 2 days before testing.
Values in parentheses are strength ratios; compressive strengths of cubes tested without intermediate drying and immersion taken as 100.

Total Moist Curing Before Test	As Removed from Moist Room*			After Drying and Absorption Tests					
				Dried at 167 deg. F.**			Dried at 239 deg. F.**		
	1:2	1:4	1:6	1:2	1:4	1:6	1:2	1:4	1:6
7 days	5160 (100)	1460 (100)	570 (100)	4580 (89)	1380 (95)	620 (109)	4620 (90)	1360 (93)	580 (102)
28 days	6490 (100)	2420 (100)	990 (100)	5570 (86)	2060 (85)	810 (82)	5100 (79)	1760 (73)	750 (76)

* Each value average of results of 12 tests.
** Each value average of results of 6 tests.

drying or drying followed by absorption has upon the compressive strength of concrete or mortar. Table 10 shows the data for the mortar tests in which the combined effect of drying and subsequent wetting can be seen by comparing the strengths of specimens treated in this manner with those of specimens which had an equal period of moist curing but with no intervening period of drying. Such a comparison shows that the strength of the weakest mortar (the 1:6 mortar tested at 7 days) was slightly increased by this treatment, although, in all other instances, the strength of the mortar was reduced by the drying and wetting incident to the absorption test. These tests were on a comparable basis, as all the mortars were wet when tested. It will be noted that drying at 239 deg. F. in almost all cases resulted in lower strengths than drying at 167 deg. F.

Table 11 shows the effect on the strength of the concretes of drying alone and of the subsequent wetting of the dried concrete. These tests differ from the similar mortar tests in that they include tests of the concretes in the dry condition and after several immersion periods. Testing the concrete in the dry condition resulted in increased compressive strengths, as compared with concrete which had not been subjected to drying. The increase averaged about 50 per cent for the concrete dried at 167 deg. F. and about 30 per cent for that dried at 239 deg. F. Those specimens which were dried and subsequently immersed gave strengths in the test 10 to 15 per cent below those of the undried concretes, with a partial recovery in strength as the immersion continued.

Table 11—Effects of Drying and Variable Immersion on Strength of Concrete

Compression tests on 2-in. cubes.

Data on composition same as in Table 7.

Concrete cured in water 14 days; one set dried 4 days at 16 deg. F., other 2 days at 239 deg. F.

Values in parentheses are strength ratios; strengths of cubes tested on removal from water taken as 100.

Condition of Concrete When Tested	Compressive Strength, lb. per sq. in.			
	5 gal. per Sack Cement		7.5 gal. per Sack Cement	
	167 deg. F.	239 deg. F.	167 deg. F.	239 deg. F.
On removal from water	5070 (100)	5070 (100)	2110 (100)	2110 (100)
Dried	7400 (146)	6380 (126)	3320 (157)	2780 (132)
Dried, immersed 30 min	5640 (111)	4880 (94)	2100 (99)	1930 (91)
Dried, immersed 3 hr	4630 (91)	4250 (84)	1900 (90)	1730 (82)
Dried, immersed 6 hr	4840 (95)	4270 (84)	2030 (96)	1780 (84)
Dried, immersed 24 hr	4850 (96)	4120 (81)	2080 (99)	1800 (85)
Dried, immersed 48 hr	4930 (97)	4300 (85)	2100 (100)	1860 (88)
Dried, immersed 72 hr	5010 (99)	4330 (86)	2250 (107)	1910 (91)
Dried, immersed 96 hr	4910 (97)	4390 (87)	2320 (110)	1950 (92)

A significant point in connection with the strength tests on the specimens which had been subjected to the wetting and drying in the absorption tests is the support which they offer to the conclusion that the absorption test is of doubtful value for concrete. It has been pointed out that drying at the high temperature of the standard test forced out water in excess of that which might be fairly classified as free water, thus creating pores to be filled in the absorption test which could not exist under any natural condition of exposure. The fact that this treatment has altered the strength is further evidence of the abnormality of the method.

Summary and Conclusions

It has been shown that as a criterion of the quality of concrete products the absorption test as now made is of little value because neither of the assumptions on which it is based is applicable to concrete. First, it does not truly indicate the porosity of the concrete as that porosity is likely to develop in service, because of the more severe drying imposed in the standard tests. Second, the porosity of the *dried* concrete which it does indicate is given in terms of the whole volume of concrete, whereas it is the porosity of the cement paste which is chiefly related to the useful properties of concrete. It is the size and extent of the pores under normal conditions rather than under the artificial conditions of extreme drying which determine the susceptibility to attack by the agencies of weathering. The size and extent of the pores are a function of the composition of the paste and its curing conditions.

A possible form of specification based on *rate of absorption* is suggested as likely to indicate more directly the quality of concrete products, especially if coupled with a strength requirement. The manufacture of a product which would comply with a specification of this type would require proper proportioning of the mix and adequate curing of the product.

The temperature at which concrete is dried prior to the absorption test, as now made, affects the results but the effect is of little moment if the limitations of the test in other ways are considered.

The advantage of the slightly lower absorption values obtained when the concrete is completely dried at temperatures below that of boiling water does not compensate for the added length of time required for drying and the likelihood of incomplete drying under those conditions. Drying at the lower temperature for a fixed period of 24 hr. is likely to lead to results difficult of duplication because of the incompleteness of drying of even small specimens in that length of time.

Discussion—Absorption of Concrete Products

Mr. Bates.

P. H. Bates, *Chairman*—Now, gentlemen, we have had three papers, all of which are concerned, either entirely or partly, with this question of absorption. I had thought we would have the specifications for cast stone presented next, and then have a discussion of the whole. I think, possibly, it is better to discuss these three papers first. I want, however, to call attention to one thing. Looking over the program, I find we have four specifications proposed for tentative adoption. I find that each one of them contains absorption values, and I find, furthermore, that each one contains a method for obtaining absorption, and, lastly, and I might say almost ridiculously, each one of them proposes a different method. Now, surely, concrete products are not so vastly different among themselves that each one must have a different method for obtaining absorption, but if they are, it would seem that it is surely a matter for considerable discussion. So the chair at this time is going to recognize himself and present some of the data in connection with absorption that I said I had for discussion in connection with the work on cast stone which was done at the Bureau of Standards.

Mr. Weigel referred to a method recommended by the Bureau, although the Bureau has not recommended any method. The method which it used in the paper was that of boiling after 48-hr. immersion. The specimens had been generally dried at 212 deg., but to supplement that work, we took specimens from a good number of the stones and instead of drying at 212 deg., we dried them at 65 deg. C., *in dry air*. Then, we obtained the absorption with only 5-hr. boiling, whereas in the paper the final value as given was 48-hr. soaking plus 5-hr. boiling. Next, these specimens were again dried at 65 deg. in dry air, and the difference in per cents between those two weights obtained. See Table I.

Let us comment a little on that difference. You will notice that, in some cases, it amounts to as much as 1 per cent, but in other cases it is lower; that is, the second dry weight is lower that the first dry weight. Those of us who have been making these absorption determinations, if we have been close observers, would have noticed that the water in which we have made these observations becomes distinctly alkaline; in other words, the water is absorbing some of the cement. The 5-hr. boiling gives a lower absorption than 5-hr. boiling plus 48-hr. soaking; or, rather, put it the other way: 48-hr. soaking plus 5-hr. boiling. But also remember that in the latter case specimens were dried at 212 deg. in ordinary air, whatever the humidity of the air in the room was, whereas in the former case they are dried at 65 deg. in dry air.

TABLE I—ABSORPTION AFTER DRYING AT 65 DEG. C. IN DRY AIR COMPARED WITH ABSORPTION AS GIVEN IN THE PAPER.

Description of Test	Specimen Number								
	1	2	3	4	5	6	10	11	13
Maximum absorption as given in paper..	8.1	8.5	7.3	9.2	8.4	11.1	13.5	11.2	11.3
Amount of water absorbed after 5 hr. boiling..............................	6.3	6.6	6.4	6.5	7.3	10.0	11.6	8.7	7.3
The difference between the first and second dry weight (see Note 1)..........	−.3	−.4	−.4	−.4	−.2	−.1	−.2	−.2	−1.

TABLE I (Continued)

Description of Test	Specimen Number								
	16	17	18	20	21	22	23	24	25
Maximum absorption as given in paper..	13.0	11.0	14.3	9.6	9.2	9.3	10.0	14.6	12.5
Amount of water absorbed after 5 hr. boiling..............................	10.5	10.5	13.3	8.0	7.9	7.9	8.4	12.6	11.1
The difference between the first and second dry weight (see Note 1).........	.0	+.1	−.1	−.2	−.1	−.1	+.1	−.1	−.2

TABLE I (Continued)

Description of Test	Specimen Number								
	26	30	31	32	33	34	35	36	37
Maximum absorption as given in paper..	12.1	9.5	11.2	10.8	8.4	10.0	13.4	10.1	11.6
Amount of water absorbed after 5 hr. boiling..............................	10.5	9.1	10.7	9.3	7.7	9.6	9.1	8.9	11.0
The difference between the first and second dry weight (see Note 1)..........	.0	−.2	−.2	−.2	−.6	−.2	.0	−.1	−.2

TABLE I (Continued)

Description of Test	Specimen Number								
	38	39	40	41	42	43	44	45	46
Maximum absorption as given in paper..	12.9	10.6	6.5	7.1	11.9	10.5	18.2	9.6	9.8
Amount of water absorbed after 5 hr. boiling..............................	11.9	9.5	5.1	5.6	11.0	8.6	17.2	8.5	7.6
The difference between the first and second dry weight (see Note 1)..........	+.1	+.1	−.9	−.6	+.2	−.2	+.1	−.2	−.3

TABLE I (Continued)

Description of Test	Specimen Number								
	47	48	49	50	52	53	54	55	56
Maximum absorption as given in paper..	11.4	11.4	11.5	12.5	5.8	4.1	11.0	14.1	6.8
Amount of water absorbed after 5 hr. boiling..............................	10.7	9.9	10.3	11.2	5.2	3.0	9.8	13.2	6.3
The difference between the first and second dry weight (see Note 1)..........	−.2	−.3	−.3	−.2	.0	−.1	−.3	+.2	−.1

NOTE 1. Three 1 x 1 x 4 in. prisms were cut from each specimen, immersed in water at 21 deg C. for 24 hr., and dried in dry air at 65 deg. C. The specimens were weighed at the end of 24 hr drying and the drying was continued until the specimens obtained a constant weight. The specimens were then immersed in water, the water brought to a boil and boiled 5 hr. They were cooled by injecting cold water, removed, excess water absorbed by a damp towel, and weighed. The specimens were then dried a second time to constant weight under the same conditions as above but omitting the 24-hr. weighing. Absorption data is expressed as the per cent of the first dry weight of the specimen.

NOTE 2. In the above figures, a + figure indicates that the second dry weight is greater than the first and a — figure, the opposite.

TABLE II—PER CENT OIL ABSORBED AFTER DRYING AT 35 DEG. C. IN DRY AIR. PER CENT WATER ABSORBED AFTER DRYING AT 110 DEG. C. IN LABORATORY AIR.

Specimen No.	Immersion													
	½ Hour		2 Hours		4 Hours		6 Hours		24 Hours		48 Hours		5 hr. Boiling in Water or 5 hr. Reduced Pressure in Oil	
	Water	Oil	Water	Oil	Water	Oil	Water	Oil	Water	Oil	Water	Oil	Water	Oil
3.........	3.2	1.0	6.3	2.1	6.7	2.6	6.8	3.1	6.9	3.9	7.0	4.0	7.3	4.4
4.........	2.5	1.4	5.0	2.7	6.9	3.7	7.3	4.4	7.6	5.9	7.7	6.1	9.2	8.2
33.........	7.4	1.4	8.1	2.6	8.1	3.5	8.1	4.1	8.2	5.1	8.2	5.4	8.4	5.8
40.........	5.7	1.0	6.3	2.0	6.3	2.6	6.4	3.0	6.5	3.7	6.5	3.8	6.5	3.9
44.........	12.3	7.0	12.6	7.2	12.6	7.3	12.6	7.5	13.0	7.8	13.6	7.9	18.2	12.4
45.........	2.8	1.2	3.8	2.0	4.4	2.6	4.7	3.0	5.5	4.2	6.0	4.3	9.6	7.0
46.........	3.4	1.9	4.8	3.0	5.4	3.9	5.7	4.4	6.1	5.1	6.3	5.4	9.8	8.3
47.........	7.1	3.9	7.1	5.5	7.1	5.7	7.2	5.9	7.2	6.1	7.4	6.2	11.4	9.3
52.........	1.0	.8	1.4	1.2	1.7	1.6	1.8	1.8	2.4	2.7	2.8	3.4	5.8	5.2
53.........	.7	.4	1.0	.7	1.2	.8	1.3	1.0	1.6	1.3	2.0	1.6	4.1	2.4
55.........	9.0	8.4	9.0	8.5	9.0	8.6	9.0	8.7	9.1	9.2	9.2	9.3	14.1	13.4
56.........	5.6	.9	6.4	1.7	6.5	2.3	6.5	2.7	6.6	3.4	6.6	3.7	6.8	4.0

NOTE. Oil absorbed calculated to equivalent weight of water expressed in per cent weight of original dry stone.

Then finally we took a small number of these stones—twelve, in fact—and dried them this time at only 35 deg. C. (and 35 deg. C. is 95 deg. F., and, of course, 95 deg. F. is a temperature that any stone is likely to reach in the summer time), but, again, in dry air. In this case, instead of using water for the absorption medium, we used oil (see Table II). Here, again, the valves are generally lower, but, again, note that they are higher in many cases. We also give here some data which were not given in the paper on the question of the rate of absorption at ½, 2, 4, 6, 24, and 48 hr., and then plus the 5-hr. boiling. The values for the water absorption were obtained in getting those given in the paper where, however, we did not indicate those for the first early periods.

Again, there are some rather outstanding points here. We took stones of average absorptions, low absorptions, and high absorptions. But it is evident that regardless of the temperature of drying or the liquid used, there are some very absorptive stones. However, the procedure used should be standard.

The oil used is what is known as 300-deg. oil, which is an oil used in the signal lamps of railways. It has a considerably higher flash point than kerosene, but is a light oil. But, when the oil penetrates so rapidly as indicated by specimens 33, 44 or 56, we can rather assume that we have very large, open pores, whereas, when we get down to some of these considerably lower ones, as 52 or 53, we know that the pore is very small, and this rather viscous material, as compared with water, penetrates into these very small pores at a very slow rate.

Although the Bureau of Standards will recommend certain specifications, we wanted a rather wide discussion on the question of absorption, the values and the methods. But, I would like to comment a little on the question of absorption myself. Why are we making the absorption test? Are we making it to determine the density? If so, why are we interested in the pores? Isn't the final answer to it that we are interested in absorption only as to the method of determining durability, and if it does not measure that, why are we making it? In other words, we ought to connect absorption up with some other facts, such as—how fast the water enters; whether the water all goes in in five minutes, whether, by actual observation and usage, we find that stone gives most excellent service, or whether we have any fault to find with it. On the other hand, if it takes 10 hr. for ¼ of 1 per cent to get into the stone, and we can find there is a definite relationship between that 10 hr. and durability, are we not interested in that relation?

Fred Weigel—On the other type of material, there has been a tendency to drift away from the absorption test because it has not been found that there is the definite relation between absorption and durability. Mr. Weigel.

P. H. Bates—Yes, the tendency is in that direction. I think the later tentative specifications of the A.S.T.M. for brick (clay brick), do not include absorption values, because after all their investigational work, they could find no relation between absorption and durability of brick. Mr. Bates.

This is about all I have to say, gentlemen, but I hope we will have a very free discussion of this matter of absorption, and just what it means. I know the committee of the Institute that has been working on the specifications for cast stone has gone into this matter in some detail, because they have proposed tentative values. I wonder if some of the members of that committee cannot give us an expression of why they adopted what they did and the reason for their method. Mr. Falcoe, can you give us a little something on that?

Mr. Falcoe.

L. A. FALCOE—I cannot give you very much excepting that that was all the data available that we could get. I thought it was really the best that we could do for the time being, realizing that there was an insurmountable amount of work to be done.

Mr. Bates.

P. H. BATES—We have a number of representatives from these other organizations who are presenting specifications here. Can't we have something from them? Mr. Allen, you are interested in this subject, I believe.

Mr. Allan.

W. D. M. ALLAN—Mr. Chairman, it seems to me that your last statement sums up the situation so well that there is very little I can add. I think there are weaknesses in the absorption tests and the method of procedure, but, as Mr. Falcoe says, from the lack of any more definite information for better method of procedure, we do not feel that either the stucco committee or the cast stone committee could undertake to work out a better procedure, since it affects so many other concrete products. It will probably have to be worked out on a broad basis with all the committees co-operating in order to determine a procedure which would be, as you point out, effective in all these different products.

Mr. Bates.

P. H. BATES—Mr. Lantz, you are presenting these specifications for concrete brick. Have you anything to say?

Mr. Lantz.

E. G. LANTZ—We have a rather new method of presenting the absorption in weight of water absorbed rather than percentages. Since brick is of a standard size, within certain limitations, we feel we are justified in placing its absorption limits on a weight basis. On the other hand, we have made no tests, it being merely a simplification of the specifications that we adopted. As Mr. Allan said, we made no changes until we could make some that would be instructive.

Mr. Christianson.

MR. CHRISTIANSON—I am very much interested in the question of absorption and I feel that, at present, we should omit all absorption requirements because we don't know the significance of the absorption test. I think that in the research work which should be done and which is being done, we should not lose sight of the fact that we do not know how the material behaves in the wall. In these absorption tests, a couple of experts sit down and decide that a brick should absorb 6 ounces of water, or 8 ounces. That does not mean anything. What happens in the wall? We want to know that. We want to know the factors of disintegration and we want to know the manner in which water travels through the wall. We want to know the manner in which moisture is taken up by the wall and how it acts in the wall and the manner in which

it is given off. The absorption tests in the past have been evolved in some laboratory or at some round table discussion. I do not think any attention has been given to what actually happens in the wall, and I think that is the basis on which we should tackle the problem.

In the brick specifications I proposed some absorption limits for brick which I think are copied from the Government specifications of 6, 8 and 10 ounces, for class A, B and C brick, class A brick to absorb not more than 6 ounces. If I were to write specifications for brick for my own house, I would put a minimum absorption requirement on the brick rather than a maximum. I know of quite a few buildings where very serious damage is being caused, evidently, from the density of the brick, clay brick having an absorption of less than 5 per cent. When you finally get them in place, a film of water is formed between the mortar and the brick, and when that film dries out, an air crack remains which draws the moisture through. It is a condition which you cannot cure. That has not been taken into consideration in any of these absorption tests.

I think there is more logic back of a minimum absorption than of a maximum absorption of six ounces for the face brick, which means about ten per cent in the case of cinder brick and about 7 per cent in the case of sand or concrete brick. Now, if a brick with 5 per cent absorption would pass these specifications, I would not have it in my house; and that is along the line I started out with, that we should study the behavior of the material in the wall. Mr. Pease correctly pointed out that the absorption test is of value only if it gives us some information on the durability. I would supplement that by saying that another factor we could consider would be moisture penetration.

In the case of cinder brick, for example, an ordinary brick that tested very close to 5000 lb. per sq. in., and that had a transverse strength of 700 extra odd pounds, would pass the specifications and would be classed as A-1 brick. However, it would suffer very considerably in a freezing test, and is not a good brick for the masons to lay in the wall. Other brick which tested 3000 lb. will be passed as a B brick in these specifications—that means, it would be permissible for outside walls and not as a face brick. Those brick, however, had an absorption of more than 8 ounces permitted for class B, and would be classed as a C brick. That was an excellent brick in every respect. It stood 42 freezings and thawings without showing any signs of disintegration, and on the jobs in which it was used, it gave excellent service. Now, a brick specification, or any specification for concrete, that uses the strength and the density as the sole criterion for quality is wrong, and I would like to see the absorption requirements dropped from all specifications, until we know just what they mean.

FRED WEIGEL—I have a few questions I wish to ask in regard to these tests. In all cases, you boil the specimens for five hours whether you dry them at 65 deg. or at 110 deg.? **Mr. Weigel.**

P. H. BATES—Yes, except these in the oil, they were not boiled. **Mr. Bates.**

Mr. Weigel. Fred Weigel—Isn't it a fact that the boiling for 5 hours, which is, naturally, at 212 or above 212, breaks down these materials that the gentleman has referred to as the "mother liquor," or the colloidal materials or the crystalline materials, whatever you call them, and isn't it a fact that they are broken down during the boiling in the same manner as they would be during the heating in spite of the fact that they are not driven out? They are broken up. And isn't it a fact that when the drying takes place, that water leaves it whether the drying is done at 65 deg. or 110 deg.?

A Member. A Member—The drying was done first, wasn't it?

Mr. Bates. P. H. Bates—Yes.

Mr. Weigel. Fred Weigel—Then, they were re-dried.

Mr. Bates. P. H. Bates—Yes. However, on the second drying, there was nothing done afterwards with the specimens. It was done simply to find whether there was any change in weight during the treatment.

Mr. Weigel. Fred Weigel—But your observation was figured from the weight after they were re-dried, was it not?

Mr. Bates. P. H. Bates—No, original dry weight.

Mr. Weigel. Fred Weigel—After they were first dried?

Mr. Bates. P. H. Bates—Yes. However, I am rather positive of a change having been made by the drying, but are we really interested in whether such is the case since, where we did not boil them, we got 9 per cent absorption?

Mr. O'Neil. Mr. O'Neil—In view of the fact that there is no accepted standard for conducting an absorption test, and in view of the fact that concrete is always exposed to an absorbing element on only one side, have you ever given any thought to making a percolation test instead of an absorption test, because the degree of permeability of concrete is in direct ratio to the amount of percolation?

Mr. Weigel. Fred Weigel—A percolation or a permeability test is not entirely satisfactory. A stone that is highly over-watered, contains a large amount of the colloidal material or the jell, as Mr. McMillan is pleased to call it, and those materials lose water and take on water rapidly during changes of temperature or changes of humidity, as this gentleman has explained. On the other hand, at normal temperatures they are fairly stable and a penetration test has no effect whatever owing to the fact that when water comes in contact with these particles they expand and close the voids entirely; if it is heated somewhat the material is broken down. An over-amount of this jell tends to decrease compressive strength; also it tends to increase expansion and contraction during changes in temperature, which causes a hair cracking, and I do not believe that a permeability test or a percolation test is therefore entirely satisfactory. I think that a combination of the two tests, which would show resistance to percolation, or to water pressure could be used in conjunction with the tests showing the amount of the jell which is an indication of the water-cement ratio.

Mr. Christianson—On the matter of the permeability test, my opinion is that it does not tell you anything as far as ordinary weather material is concerned. I come back to the statement that we must find out how the material behaves under actual exposure, how the water travels through a wall. Now, when we take the outside of a building which is exposed to a rainstorm, that water is not forced through under 30 or 40 pounds pressure; it is forced into the surface depth of the wall perhaps a half an inch by the force of the wind, and from there it travels by capillary attraction. A permeability test gives no information on the behavior of the material in the wall. You can take a smaller piece of concrete and you can pour water right through, you do not need any 30 pound pressure, but you can put it in the wall, and if the wall is of the proper thickness and properly constructed, no moisture will come through. Therefore I do not think that a permeability test is of any interest unless we are concerned with a material that is subjected to water pressure. Mr. Christianson.

Mr. Levin—I believe that Mr. Christianson is absolutely correct in all of his statements. I believe that we are interested in knowing the service that a wall is to perform. The strength and probable absorption may indicate something, but if that wall is built of light-weight aggregates, a strong absorption requirement might cause the manufacturer of such bricks to reduce the density and thereby improve the efficiency of the brick when it comes to absorbing, some acting as heat insulators and in resisting freezing. I believe that the absorption requirement for concrete requirements, should be eliminated from the standard specifications. Mr. Levin.

A Member—I do not want to prolong this unnecessarily but I have been brought up in a laboratory, and I agree with Mr. Christianson that this absorption test was something that originated in the laboratory, and the reason for it was probably for something definite to do; not only was it apparently definite, but it was something easy to do. Of course, there are a lot of things you could overlook, the size of the specimen and all that sort of thing, which make a tremendous difference in the results you get but showing the absorption in percentage of weight of material, really doesn't mean anything, because you have got to go back to the basis on which you operate. It is something like the cement-briquette test, which, in spite of my training, I have always been bolshevistic about. It never has been, in my opinion, any good. I mean to say it is not a definite measure of the quality of the cement. The test is theoretically wrong, and, practically, it is something like the absorption test; the test on the low end will show high absorption and low strength, and, then, on the upper end will show fairly high strength and low absorption. When you get right down to comparing something that will stick together, they mean absolutely nothing, and I will be very much in favor of seeing this absorption requirement dropped, because I think we do not have an adequate interpretation of it. A Member.

W. D. M. Allan—The absorption test on certain kinds of concrete products has been a very effective way of regulating quality. It may not be that we have known exactly how to interpret it, and it may be Mr. Allan.

that we can improve it a great deal, but, as I recall we have found from experience that dense, strong concrete resists the attack of acids and the absorbing action of water better than coarse, weak concrete. Now, rather than go to the other extreme of dropping this absorption test completely and throwing out all that we have built up, I would sooner see it analyzed, critically analyzed, and some middle ground, possibly, picked out. Drop the absorption tests from those products where it does not seem to be necessary, and leave it in those products where it does seem to be necessary. I am a little bit afraid to go from one extreme to the other, not having any more information for throwing out the absorption tests than we have now for keeping it in. I think that the majority of the cast stone men, for instance, will agree that very seldom do you have trouble from cast stone that is strong and dense. A cast stone made with a grade of aggregate that makes it more dense, whether dried or boiled for five hours—the particular method is not so important—does not absorb much water, does not have as much fluorescence, does not discolor, and is, generally, a better quality of concrete and the kind of stone that has aided the cast stone industry in building a reputation.

We may not be able to explain just why the absorption test is of value, but it certainly, to my way of thinking, is an indication of a satisfactory condition in classifying the product. My one thought in speaking now is just a caution against going to the other extreme from where we are now without any more reason for dropping the absorption test than because we cannot interpret it. First, I think, we should find out how to interpret it.

Mr. Bates. P. H. BATES—Gentlemen, we will have to stop the discussion because time is short, but I think, possibly, that Mr. Allan's summation is something for all these various committees working on these problems to bear in mind. It seems to me that these various committees should present more data than they have so far done. I think if the committees will bear that in mind the membership as a whole can decide more satisfactorily whether we should have 8 per cent absorption or 10 per cent or zero or what and how it should be determined.

Mr. Gilkey. HERBERT J. GILKEY*—The data of this paper and the clearness of their presentation are such as to contribute much to a better understanding of many things about the behavior of concrete.

In timeliness and sound practical value, the writer sees a certain common ground between this paper and that by the author's co-workers, Gonnerman and Woodworth. Both papers aid in no small degree in the removal of looseness and vagueness regarding certain important points in concrete thought and practice. The paper confirms, in a full and convincing manner, observations that the writer has made in the course of following up false leads. The two major instances will be cited for whatever they may add in the way of confirmatory evidence.

1. In common with many persons, no doubt, the writer visualized two states of water in concrete. (*a*) Free water, inert and occupying pore

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space. (*b*) Chemically combined water, present in the cured portion of the concrete as water of crystallization. Class (*a*) was mentally subdivided into the water occupying the large pores which could be easily driven off and that in the small passages that was held by strong capillary adhesion. These may be all. It is quite possible that the sole affinity of the amorphous materials for water is that of capillarity. Be these things as they may, the writer conceived the following premise and proceeded to seek for experimental verification of it.

"Since the cement and the water are the only chemically active ingredients in concrete, it would appear that the bond between the cement and the water should be strictly chemical and that the water held by the inactive part of the concrete or mortar (assuming absorption by aggregates to be negligible) would be mere filler, easily driven off. It therefore seemed reasonable to expect heating at some temperature just above boiling, or possibly below it, to drive off practically all the water that was not held by the cement. The part not driven off would be, according to this theory, proportional to the amount of cement in the mixture and might therefore serve as an accurate and simple criterion for ascertaining how much cement had been used in concrete that had hardened." Probably the writer is but one of many who have followed the same logic.

To check the theory mixtures ranging from the very-wet to the unworkably-dry were made of proportions varying from neat cement to 1:5 standard sand mortar. Standard sand was used because of its unquestioned inertness and non-absorbability and also because its coarseness eliminated the possibility of small capillary passages in the aggregate itself. Specimens were oven dried at two temperatures, approximating 160 deg. and 225 deg. F. Other samples were also air dried in the laboratory and weight data taken. Seven and 28-day moist curing preceded the different drying processes. Both air-dired specimens and those oven-dried at the lower temperature were later dried at the higher temperature and the losses noted.

The findings were exactly those recorded in the paper. None of the data appeared to bear any usable relationship to the amount of cement in the mixture.

Air drying does not furnish a constant reference datum of dryness and it is doubtful if oven drying does so. Constant weight in air is only attained for air of constant humidity. As humidity of the air varies the concrete will give up or take in moisture. It never seems to reach any fixed state of dryness that it can maintain in a varying atmosphere. This is also true of oven drying to some extent, as is shown in Mr. Wilson's tabular record of results for the two temperatures given. The drying to constant weight at 239 deg. gives a greater loss than at 167 deg. F. From other tests it seems evident that the moisture content at constant-weight dryness will always become somewhat less as the air humidity is lowered by the application of heat. Oven-dried specimens always slowly take on some weight after removal from the oven, even though the air to which they are subsequently exposed is relatively dry.

2. The other false start is with respect to the wet or dry condition of specimens at test. It is well established fact (although too often one that is overlooked) that the strength difference between being air dry or saturated at time of test is from 15 to 30 per cent in favor of the dry concrete. Since dry concrete is always potentially wet concrete, the wet strength appears to be the fairer one to use as standard, although for present purposes that is beside the point. The important thing is that specimens that are having their strengths compared should not have this extra variable injected into the findings. Incidentally the procedure in the tests reported in the present paper was admirable in this regard. Very logical measures were adopted to attain strength results that were probably as fully and fairly comparable as it is possible to attain when both the curing and moisture variables are present at early ages. The curing variable becomes much less important as the age advances. For specimens of 6 mo. or more in age a few weeks of moist curing, more or less, has little effect upon the strength. For such concrete the wet-dry variable can be easily and reasonably evaluated.

With this important point in mind, it is evident that the saturated condition at test is one that is easily attained. Air drying is too slow. More detailed attention is accorded this phase in connection with a discussion of the Gonnerman and Woodworth paper. Oven drying appeared to be the possible way to obtain a dry common denominator or datum. Tests were made, but the forced drying did not affect the specimen in the same way as the more gradual air drying. In the writer's tests (made 3 years ago) the oven-dried strength was about the same as the saturated strength. The gain due to drying out was lost in the drying process. Re-soaking after drying showed that the loss was real, although a period of resumed moist curing had a tendency to repair the damage.

These findings are all in excellent accord with those that the paper reports, except that oven drying did produce very high increases in strength for specimens tested dry although the specimens were basically weakened as indicated by the reduced strength at re-saturation. Table 11 gives very valuable quantitative information on these points. It shows the greater injury that results from drying at the higher temperature and also the substantial recovery when the re-immersion was continued for longer than was necessary to attain practical saturation. This is no doubt due largely to the gain from resumed curing. In Table 10 certain apparent discrepancies appear for a number of the 7-day specimens. The process of drying plus re-saturation appears to have left the specimen stronger than it was in the original saturated condition. In no recorded instance was this true for the 28-day specimens. It seems very probable that the gains come from the extra curing that the specimens got in the course of being re-soaked. At ages as early as 7 days the rate of curing is so rapid that every day of favorable curing environment means a relatively large strength change.

Concluding Remarks—The paper bears out the statement that the writer has often made, that for all ordinary sizes of laboratory specimens,

practical saturation will be attained in 24 hr. or less. In soaking specimens (especially those of rather early age) to bring them to the standard wet condition for testing, they should not be immersed for longer than 24 hr. or the resumption of curing process will tend to build up the strength. There is also a slow increase in weight due more, perhaps, to the resumption of chemical activity than to a mere mechanical filling of pore space, which is all that the saturation should aim to accomplish.

The writer fully agrees, with the author, that the amount of absorption itself (if not excessive) gives little insight into the relative excellence of concretes. He further agrees that the rate of absorption might prove to be a very useful and usable criterion.

Specification for Cast Stone*

Report of Committee P-3

The adaptability of cast stone to various architectural requirements has during the past ten years given it a prominent place in the field of building materials.

The pioneers in the manufacture of cast stone were trained not only in the fundamentals of architectural design but also in the fundamentals of good concrete practice and the stone which they produced was for the most part of high quality. In the development period that followed, many manufacturers entered the cast stone field who were trained either in the architectural details or in the fundamentals of good concrete, but not in both, and this often resulted in stone of unsatisfactory quality. Considering cast stone as a whole, more emphasis has been laid on architectural details than on the quality of the concrete used. Without underestimating the importance of architectural details in cast stone, it must be recognized that all the beauty of surface treatment, fine modelling and the most accurate interpretation of architectural motifs will be lost unless the concrete in which they are expressed is of durable quality.

The importance of standards of quality for cast stone was recognized at the 1928 convention of the American Concrete Institute through the creation of a new committee to study the problems involved. A survey conducted by the committee indicated that the problems affecting the use of cast stone fell under three classifications: (1) Quality of the concrete; (2) surface treatment of the stone; (3) methods of setting.

It was the unanimous opinion of the committee that its attention should first be directed to the preparation of specifications which would regulate the quality of the concrete from which cast stone is made, and then to study the other problems as soon as possible.

The committee has proceeded on the basis that the values established in the proposed specification should be higher than the present average quality of cast stone as indicated by the tests made by P. H. Bates and the committee. The committee believes that it is justified in assuming this position because, as was pointed out earlier in the report, many cast stone manufacturers have centered their attention on architectural details at the expense of the quality of the concrete. Tests have shown a range of from 1500 to 10,000 lb. per sq. in. in the compressive strengths of cast stones. The committee wishes to emphasize that the

* This specification was adopted by the convention as Tentative Standard, P3–A–29T

leading manufacturers have not only perfected architectural details but are producing a quality of concrete in their cast stone that exceeds the values set in this specification.

After completing as much as possible of its work by correspondence, the committee held a meeting in New York City on October 27, at which time the general form of the specification was adopted and values established. The committee then conducted a series of tests on 26 varieties of cast stone collected from 19 manufacturers. The tests were made to determine the fairness of the values selected and the practicability of the test procedure specified. At a meeting of the committee held in Washington December 17 and 18, the results of these tests were considered and necessary revisions made in the proposed specification.

In adopting compressive strength and absorption as measures of the quality of the concrete in cast stone, the committee has followed the precedent established by specifications for other forms of concrete. At this time there do not seem to be any measures of quality more reliable than these and it is the conviction of the committee that the requirements embodied in the proposed specification are the minimum requirements which will ensure high quality in cast stone.

The work of the committee has been greatly facilitated by the cooperation and assistance given by the Specification Committee of the Association of Cast Stone Manufacturers. The committee is especially indebted to P. H. Bates for the valuable information supplied by him through the Bureau of Standards' investigation of cast stone.

The following specification is presented to the American Concrete Institute for adoption as a tentative standard specification on cast stone:

I. General

(1) The term "cast stone" shall be understood to mean a building stone manufactured from portland cement concrete, precast and set in place as trim or facing on or in buildings and other structures.

(2) The minimum average compressive strength of cast stone at the age of 28 days or less shall be 5,000 lb. per sq. in. when tested as 2 x 2-in. cylinders or 2 x 2-in. cubes in the manner hereinafter specified.

(3) At the age of 28 days or less the average absorption of cast stone shall be not less than 3 and not more than 7 per cent of water by dry weight of the specimen when tested as 2 x 2-in. cylinders or 2 x 2-in. cubes in the manner hereinafter specified.

(4) All aggregate used in the manufacture of cast stone shall be of known durable quality.

II. Selection of Specimens for Testing

(5) Specimens for both compression and absorption tests shall be cut from stone as delivered on the job or from regular stock in the yard. Samples from which specimens will be cut shall be selected by the purchaser or his representative.

(6) Specimens of faced cast stone for compression tests shall be cut in such a manner that the specimens are composed of approximately one-half of facing and one-half of backing material. Specimens shall be tested in the position in which the cast stone is laid in the wall.

III. METHODS OF TESTING

(7) Not less than 3 and preferably 5 specimens shall be required for each test. In the event of failure to meet requirements in the first test, the test may be repeated on a second set of specimens.

(8) Specimens for absorption test shall be dried at a temperature between 215 and 225 deg. F. until the loss in weight is not more than 0.1 per cent in 2 hr. of drying. They shall be weighed and then be submerged in water at a temperature of between 60 and 80 deg. F. for 24 hours. The specimens shall then be removed from the water, the surface water wiped off with a damp cloth and the specimens weighed. The percentage of absorption is the difference in weight divided by the dry weight of the specimen and multiplied by 100.

(9) Specimens for strength test shall be dried at a temperature of between 215 and 225 deg. F. until the loss in weight is not more than 0.1 per cent in 2 hr. of drying.

(10) If bearing surfaces of specimens for strength test are not smooth, they shall be made so by grinding. If they cannot be ground to a smooth surface they shall be capped with a mixture of ½ part portland cement and ½ part plaster of paris, which shall be allowed to harden at least 5 hr. before the test. The cap shall be formed by spreading the capping material upon a plate glass and pressing the specimen firmly on it making the cap as thin as possible.

(11) Load shall be applied through a spherical bearing block placed on top of the specimen in a vertical testing machine. The dimensions of the bearing block shall be the same, or slightly larger, than those of the test specimen.

(12) Load shall be applied uniformly and without shock. The speed of the moving head of the machine shall be not more than .05 in. per min. when the machine is running idle.

(13) Specimen shall be loaded to failure and the unit compressive strength calculated in pounds per square inch. The type of failure and appearance of specimen shall be noted.

In proposing that the minimum compressive strength of cast stone at an age of 28 days or less be 5,000 lb. per sq. in., the committee realizes that upon first consideration the requirement may seem somewhat high. However, it must be pointed out that the compression test is made on a comparatively small specimen which is completely dry—two conditions tending to produce higher strengths in concrete. Moreover, tests have definitely shown that 5,000 lb. per sq. in. is a fair value for the compressive strength of cast stone.

The ability of cast stone to remain clean and free from discoloration is one of its greatest advantages. It is important that such an advantage be maintained and improved and to that end the committee has adopted absorption as a measure of this desirable property. The committee is aware of the questions that have been raised as to the correctness of the absorption requirements and present methods of making the absorption test. However, it is firmly convinced that at this time no more reliable measure or procedure is available.

This report has been submitted to letter ballot of the committee, which consists of 9 members, of whom 9 have voted affirmatively, none negatively, and none has refrained from voting.

L. A. FALCO
Chairman, Committee P-3

Mr. Walker. C. G. WALKER—In order to bring this matter to a head and to get the concensus of the meeting, I move that the report of the committee be accepted and that the proposed specifications for cast stone be adopted as a tentative standard. In making this motion, I realize fully the inadequacy of the present absorption test, and I make it with the understanding that the committee shall continue its efforts to devise some more satisfactory means. It seems to me that the best way to keep the question before us is to keep the absorption requirement in our specification.

Mr. Weigel. FRED WEIGEL—From the tenor of the paper that I just read, it would naturally follow that I would be opposed to the specifications just presented by the committee. However, I had the privilege of sitting with the committee yesterday morning for nearly two hours and became acquainted with a great many of the difficulties that it has encountered in working up these specifications. It has not been an easy job which has been required and the committee has done its level best and feels that the standards which it has established are as nearly fair and comprehensive as is possible to make them at this time.

The question of absorption seems to be the sore spot. I think it has worked some hardship on some of the cast stone manufacturers, especially the wet-mix stone manufacturer. However, that difficulty can be overcome to a very great degree by the use of a small amount of soap base or metallic soap as has been already suggested, and we have already pointed out that the heating test, while it is not indicative of the actual mechanical absorption, does have an advantage and is quite valuable in laboratory research. In view of these things, I want to express myself as being in favor of accepting the recommendations of the committee on the condition, and on this condition only, that the committee, or another committee working in conjunction with committee P-3, be instructed to continue the investigations to try to arrive at a more comprehensive test for absorption and the other factors concerning quality of cast stone.

Mr. Bates. P. H. BATES, *Chairman*—The motion has been made that this report of the committee on proposed specifications for cast stone be accepted as a tentative standard. All in favor will signify by saying "Aye," opposed "No." Unanimously carried.

Mr. Meacham. JAMES A. MEACHAM (*By Letter*).—Referring to the proposed specification for cast stone as reported by committee P-3, we note that the minimum average compressive strength for cast stone at the age of 28

days has been established at 5000 lb. per sq. in. when tested as 2 x 2-in. cylinders or 2 x 2-in. cubes in the accepted manner.

Although the committee has adequately supported its decision in placing the required strength of this material at a comparatively high level, we feel the subject merits further discussion.

Cast stone is used occasionally in connection with monolithic concrete work as for example the sill or coping over a concrete wall. In such cases the requirement for strength in the case of the cast stone would appear high compared with that ordinarily expected for concrete in general and more especially so since the items in question are not essentially structural in character. A reference to field-cast concrete stone is not particularly relevant to a discussion of the products covered by the committee's proposed specification. However, it does occasionally prove necessary, or at least advisable, to produce certain items of cast stone at the job. The casting of these items in concrete which would consistently show the high unit strength stipulated would involve some difficulty and in view of the use to which the material is ordinarily put seems hardly to be justified.

The point is raised without criticism and simply as a subject for discussion, bearing in mind that the committee has no doubt adequately covered all possible angles of the question as its recommendations were developed.

HERBERT J. GILKEY*—Several points will be discussed in the order in which they appear in the preprinted specifications. **Mr. Gilkey.**

1. *Dimensions of Test Specimens.* (Sec. I, Art. 2).

Concrete practice has standardized on the cylindrical compressive specimen with height twice the diameter. It is true that the small cube (2 in.) is often used more or less interchangeably with the 2 x 4-in. cylinder. The cube of any size has very obvious advantages and disadvantages. The most obvious and important advantage is that the specimen may be cast against two plane surfaces and thus good testing ends may be secured without the necessity of capping or bedding. The specimen may be tested against a pair of sides and the somewhat irregular top surface need not be used for bearing.

The disadvantages are several. (*a*) The relative shortness does not permit normal action over any of its length. If the failure is conical, height is not sufficient for the independent development of either end cone. The two overlap. The end restraint from lateral friction is considerable and probably affects the action materially. (*b*) The corners are objectionable. It is certain that the stress distribution at the corners must be materially different from that elsewhere. (*c*) A member as short as the cube is materially stronger (in spite of the corner weakness) than is the higher specimen. (*d*) If stress-strain data are desired for either modulus of elasticity determinations or for Poisson's ratio, the short specimen is especially undesirable and the findings are likely to be materi-

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ally in error if determinations on a longer specimen without corners are assumed to be correct.

The square prism of greater height than width would remove some of the objections but not all. The corners remain. The longer prism could be cast with plane ends, just as could the cube, by testing against a pair of sides as poured. This cannot well be done for a cylinder because of the difficulty of side casting. There are objections for the prism or cube. Most elastic materials, whether they be unstiffened concrete or molten metal, will segregate by gravity action in the interval during which they are soft. Thus the top of a casting or an ingot will materially differ in quality from the middle and bottom. (In this case heat also has its bearing on the phenomenon.) In a concrete casting, solids settle and water and fine particles rise until the initial stiffening occurs.[1] This gives a series of parallel horizontal layers of unequal quality and strength. If tested as poured, the top will be weaker than the bottom but the resistance to load will be concentric. If tested on the side as poured, the center of resistance is not the axis of the specimen and there is eccentricity of resistance which is identical with eccentricity of loading. Any text on mechanics of materials may be consulted to demonstrate the weakening effect of even very slight eccentricities. Some indication of its practical measure is apparent from published data on side-poured specimens.[2,3]

The 2 x 2-in. cylinder that is proposed is free of corners but retains the objectionable shortness and need for capping when it is cast in that shape.

So far as ease in procuring specimens from blocks of cast stone is concerned, the relative merits are mainly a question of the more available equipment. In some respects the advantage lies with the cube or prism which may be sawed. The procedure is tedious and costly enough in either case.

For purposes of comparison of test results, the cube is better if cast stones are to be compared with natural ones, since the cube has been the type of specimen employed in most compressive tests on stone. In the field of concrete there is a mass of comparative cube and cylinder data and there are generally accepted reduction constants for strength conversion.[4,5,6,7]

How invariable these may be expected to be for different mixtures, etc., is a question not fully answered. For the short cylindrical core proposed, there are relatively few data for conversion either to the cube or to the standard cylinder. This leaves a real question as to the advisability of adding one more specimen type to a field already split between two.

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[1] See Water Gain and Allied Phenomena in Concrete Construction, *Engineering News-Record, Feb. 10, 1927.*
[2] *Proc.* A.S.T.M., Vol. 25 (1925), Part I, Table 1, p. 200.
[3] *Proc.* A.S.T.M., Vol. 25 (1925), Part II, pp. 253–255.
[4] H. F. Gonnerman, Bull. 16, Portland Cement Association, Lewis Institute. In the back of this bulletin a good bibliography is given.
[5] *Proc.* A.S.T.M., Vol. 25 (1925), Part II, pp. 237–255.
[6] *Proc.* A.S.T.M., Vol. 25 (1925), Part I, p. 201.
[7] *Proc.* A.C.I., 1923, pp. 191–197.

The writer is inclined to feel that cast stone is essentially concrete and that the standard specimen should agree with that used and standardized for concrete. Because of height and corner considerations he feels that the retention of the cube as a permissible duplicate standard type is not desirable. It is true that any consideration of stress-strain relations and dependent properties is not at present pertinent so far as cast stone is concerned. Nor do casting problems have any direct bearing upon the sawing or drilling of samples from blocks. But these things are all vital in the general selection of the concrete test specimen that is to be considered standard. It is essential that the standard specimen be such that it can be satisfactorily cast and that reliable stress-strain test data may be secured when needed or desired. It is further to be desired that the standard specimen be such that it can be duplicated by either sawing or drilling. Uses are so varied that this last condition can sometimes be met and sometimes not. In the present instance relative feasibility will have to be the controlling factor after the conflicting elements have been weighed and pitted one against another. It is recognized that a core 4 in. high is relatively costly as compared with one 2 in. high. It is also recognized that the size of aggregate commonly used is such that any diameter under 2 in. is probably unwise. Thus, while a considerable sacrifice is justified in the interests of standardized test specimens, the writer himself is not fully decided in his own mind as to the size and shape of specimen that should be adopted in these specifications. The conflicting elements have been reviewed at some length only because it was thought that in a rather thorough summary some point not already fully weighed might be unearthed and that the final decision might be a better one because of it.

2. *Permissible Absorption.* (Sec. I, Art. 3)

As has been so well brought out in the paper by Raymond Wilson, the absorption test is of doubtful value in so far as any definite bearing on the properties of the concrete is concerned. Nevertheless, limiting values such as are specified are desirable. The rate of absorption is, as Mr. Wilson has suggested, much more indicative of age, richness or general excellence of the concrete and might prove to be a test worthy of development.

The detailed procedure to be followed in any absorption test is important and must be standardized. The saturated condition is easily obtained, but dryness is only relative and the datum for judging dryness is arbitrary and needs to be selected with some care. The writer is inclined to the belief that the suggested temperature of 215 to 225 deg. F. is too high because of the amount of weakening that results from such highly forced drying. On the other hand, air drying is too slow and variable. Perhaps oven drying at more moderate temperature is the proper compromise.[8] Mr. Wilson's paper, especially Table 10, supplies valuable data

[8] It will be noted that this suggestion agrees with that on the second page of "The Physical Properties of Commercial Cast Stone" by Tucker and Walker which appears elsewhere in this volume. The further data there mentioned may indicate the solution.

on this phase. The writer's discussion thereof considers the matter in more detail. The question is not whether oven drying seems to increase the dry-tested strength, but does it increase it as much as more gradual drying would. If the re-saturated specimen, after oven drying, is weaker than the initially wet-cured wet-tested (standard) specimen, it is evident that either the drying or the extra soaking damaged or weakened the specimen. From other tests by the writer, it appears that the forced drying rather than the extra soaking is the cause of the reduced strengths of Table 10 (Wilson's paper). The increased 7-day strengths in two cases are probably due to added or resumed curing upon final immersion. (See the wirter's discussion of Wilson's paper for more detail.)

3. *Selection of Specimens for Testing.* (Sec. II, Art. 6)

If the facing and backing concretes are of about the same strength and stiffness, it is evidently proper that a specimen be about half in half as specified. If not, there will be eccentricity of resistance which will affect the test results exactly as would eccentricity of loading as was mentioned earlier in the discussion. The apparent compressive strength would be representative of neither backing nor face and might even be weaker than either because of the bending introduced by the off-center resistance. If the facing be too thin to test independently, just how to check fairly its strength is a problem. The strength, as a measure of load carrying capacity, is of interest mainly for the backing and it would seem that specimens entirely of backing should be tested. Uusing strength as one criterion of sound durable concrete (as is reasonable), makes its employment in connection with the facing even more important than with the backing. In cast stone work the durability of the exposed surfaces is invariably of greater concern than is the strength of the backing. Adequacy for the latter is much the simpler to secure. If the facing be too thin to render suitable specimens, it would seem advisable to depend upon other criteria for judging its quality. The writer is inclined to brand the "half-in-half" core an unrepresentative and therefore a misleading and rather valueless criterion of strength.

4. *Retests.* (Sec. III, Art. 7)

If there is reason for doubt as to the correct and adequate test procedure, then the test should be repeated, whether the specimens passed or failed. The writer considers the provision for a retest, in case of failure to pass, to be thoroughly unsound. If the purchaser is willing to encounter the risk of getting some inferior material because the three or five samples selected happened to be above average, then the manufacturer should be equally willing to have his product stand or fall upon the same basis. This assumes, of course, that honesty and impartiality have been used in the selection of the samples. Carelessness in sampling or attempting to pick either the worst or best of the lot for test will make a farce of any specification. The writer knows of no more masterly treatment of this

phase than "The Enforcement of Specifications," by the late Charles B. Dudley.[9]

5. *Condition at Test* (*Wet or Dry*). (Sec. III, Art. 9)

The oven-dry condition at test is specified. One reason for doing this is given on the third page of the Tucker and Walker paper on "The Physical Properties of Commercial Cast Stone" (in this volume). The contention is that since water will be used in the preparation of the samples from larger blocks, drying is essential to bring all to a determinate condition for test.

The writer feels that the above reason is not of sufficient weight to warrant the use of a dry specimen for strength determinations. The most easily attained standard condition is that of saturation. Twelve to twenty-four hours of immersion insures that. It is the standard condition for concrete compressive tests. The test of a dry sample is non-standard, artificial, more difficult and costly to perform and is misleading in its inference. The results are not comparable with those from other standard concrete tests. The fact that the stone, as generally used, is dry in service, is no valid argument for a dry test. Tests are to determine relative merits and if they will accomplish this it is not essential that they duplicate practical conditions of use. The properties for the determination of which tests are made need only parallel those necessary to give satisfactory service. For a further consideration of the objection to dry tested specimens for purposes of strength comparisons the reader is referred to the writer's discussion of the Gonnerman and Woodworth paper which appears in this volume of the *Proceedings*.

6. *Bearing Surfaces for Strength Tests*. (Sec. III, Art. 10)

A properly ground surface is always a good one for test. If plaster of paris were to be used in lieu of the ground surface, as provided in Art. 10, and the dry specimen of Art. 9 were employed, the moisture from the plaster would be a complicating factor. This difficulty could be avoided by requiring that the specimen be painted with shellac on all surfaces that might come in contact with the moist plaster, as is commonly done in the testing of brick, tile, etc. Of course, there are other protective means. If the dry-test specimen is to be retained, the specification should, by all means, cover this detail.

Grinding is often difficult and plaster of paris is always messy, and there is furthermore some doubt as to its entire adequacy. It is very common practice to avoid the use of plaster by the substitution of such bedments as celotex. This procedure is so very simple and clean and has so much in its favor that it would appear desirable that the committee should, by all means, investigate its merit in comparison with ground and plaster ends. It should then insert a clause that would either control or prohibit the use of such types of bedment.

[9] *Proc.* A.S.T.M., Vol. VII (1907), pp. 31–32.

7. *Spherical Bearing Block.* (Sec. III, Art. 11)

The Tucker and Walker tests for physical properties used the spherical bearing block below the compressive specimen (sixth page of paper) instead of on top as specified. This introduced only a minor variable for carefully regulated work. The top block is always preferable because it agrees with the usual standard requirement and is less likely to produce cramping and bending stresses in the specimen.

8. *Closing Remarks.*

It is evident that if the proportions of the test specimen were altered as suggested in (1) of this discussion, or if the saturated condition at test were to replace the oven-dry condition of Art. 9, as is suggested in (5), the specified minimum strength would have to be materially lowered. There is no reason why this should not be done. The present 5000 lb. per sq. in. for dry short specimens means much less for a longer, saturated specimen. It gives an entirely false impression as to the quality of concrete in the cast stone. Cast stone has nothing to gain by such misrepresentation.

Although freezing, fire and weathering tests are costly and difficult to perform and are not as yet any too well linked to natural weathering action, the desirability and impotance of some test that will give an index of probable surface durability, is unquestioned. Few of the problems of cast stone rank with that. The committee will do well to continue its investigations with a view to finding and incorporating in the specification the best possible test for this all-important factor. The flexural test should be further investigated.

Although this discussion may appear to be super-critical and more destructive than constructive, it is not intended that it be so. The writer is very glad to see definite specifications for cast stone being formulated because the field for this phase of concrete is a large and important one. He is anxious to see the specification cover the ground in the best possible manner and to this end he assumes that the committee welcomes the mention of every point that might be in any measure suggestive to it, regardless of what the committee's final judgment might prove to be. It is to that end that both petty and major questions have been raised, sometimes with, and sometimes without, any tangible suggestion as to means for betterment.

Tentative Specifications, Finish Coat Portland Cement Stucco*

Report of Sub-Committee 1 on Stucco of Committee C-3—Treatment of Concrete Surfaces

Introduction

The present Sub-Committee 1 on Stucco of Committee C-3 was appointed in May, 1928, to study recommendations and specifications for the preparation and application of portland cement stucco. For some time it had been recognized that the Standard Recommended Practice for Portland Cement Stucco, C-3a-23, adopted by the American Concrete Institute, April, 1923, was insufficient to cover changes in stucco practice brought about by the demand for more highly textured surfaces. Finish coat material for many of the new textures could not be applied if made in accordance with the old recommendations, which called for proportions of 1 part of portland cement to 3 parts of aggregate, to which could be added hydrated lime not to exceed 1/5 the volume of cement, and the aggregate was to be graded from 0–8. Failure of the C-3a-23 recommendations to produce a satisfactory workability left manufacturers without a guide and led to general disregard for standards of quality. Analysis and investigation showed that factory-prepared and job-mixed stuccoes varied widely with regard to proportions and physical properties. Proportions varied from 1:1½ to 1:4; compression strengths from 300 to 4,000 lb. per sq. in.; absorptions from 5 to 20 per cent. All of these stuccoes were advertised and sold as portland cement stuccoes, yet many of them had few, if any, of the properties of portland cement mortar.

Architects and builders had no way of telling whether so-called portland cement stuccoes could be relied on to give results expected from portland cement mortar. Reputable manufacturers and others interested in increasing the use of stucco had no standard by which a portland cement stucco could be measured.

With competition keen and the only requirement for portland cement stucco being that the name portland cement appear on the bag, plasticity and covering capacity became the chief considerations when making sales. Plasticity agents were frequently added in quantities of more than 50 per cent of the volume of cement, without regard to the effect of these admixtures on durability.

Because of the recent general failure of a widely-used competitive exterior stucco material, builders are critical of stucco. The recent tend-

* This specification was adopted by the 1929 convention as Tentative Standard, C3-C–29T.

ency of some portland cement stucco manufacturers to disregard those properties of portland cement mortars that insure permanence, makes it imperative that steps be taken to safeguard the future of the product. To that end the following proposed specification for finish coat, portland cement stucco is presented for adoption as a Tentative Standard Specification:

I. General

(1) The purpose of these specifications is to establish minimum requirements for finish coat portland cement stucco.

(2) The term "finish coat portland cement stucco" shall be understood to mean a portland cement mortar used to cover or decorate preceding coats of portland cement stucco or other suitable bases on exterior walls and surfaces exposed to the elements.

(3) The minimum average compressive strength of finish coat portland cement stucco at 28 days of age shall be 2000 lb. per sq. in. when molded and tested as 2-in. cubes in the manner hereinafter specified.

(4) Finish coat portland cement stucco shall not absorb more than 10 per cent of water when tested as hereinafter specified.

(5) Finish coat portland cement stucco shall not contain more than 35 per cent by weight of the whole sample of material passing the 100-mesh sieve.

(6) If pigments are used, they shall be pure mineral oxides guaranteed by the manufacturer to be of uniform quality and proof against action of lime and sun.

II. Method of Making Specimens

(7) Finish coat stucco to be used in making specimens for all tests shall be mixed to plastering consistency. The approximate amount of water required to mix any stucco to plastering consistency shall be indicated by the manufacturer of that stucco.

(8) In making specimens, molds shall be filled in two layers, each layer being lightly puddled with the finger. Stucco shall be left heaped on molds and be stuck off at the end of 3 to 4 hrs. Immediately after molding, specimens shall be covered with moist burlap for 24 hr., then removed from the molds.

(9) After removal from molds specimens shall be immersed in water for 6 days and thereafter stored 21 days in dry air of the laboratory at approximately 70 deg. F. Specimen shall be tested at age of 28 days.

III. Method of Testing

(10) Not less than 3 and preferably 5 specimens shall be required for each test. In the event of failure of the first set of specimens, the test shall be repeated on a second set of specimens.

(11) Absorption tests shall be made on 2-in. cubes. After being cured, as provided in paragraph 9, they shall be carefully weighed and

then completely submerged in water at a temperature of between 60 and 80 deg. F. for 24 hr. Specimens shall then be removed, the surface water wiped off with a damp cloth and the specimens quickly weighed The percentage of absorption is the difference in weight divided by the dry weight of the specimen and multiplied by 100.

(12) Specimens for compression test shall be tested in a vertical testing machine of not exceeding 50,000 lb. capacity. Load shall be applied through a spherical bearing block placed on top of the specimen. The dimensions of the bearing block shall be the same or slightly greater than those of the specimen.

(13) Load shall be applied uniformly and without shock. The speed of the moving head of the testing machine shall be not more than 0.05 in. per minute when the machine is running idle.

(14) Specimens shall be loaded to failure and unit compressive strength calculated in pounds per square inch. The type of failure and appearance of the specimen shall be noted.

Your sub-committee held two meetings in New York City on October 26 and December 19. At the first meeting the form of the specification was developed and tentative values set. The committee then obtained 23 samples of factory-prepared stucco and tested them in accordance with the provisions established in the proposed specification. At the second meeting the test results were carefully studied and the specification revised.

In preparing the specification, the importance of plasticity was given careful consideration. Through comparing test results and the known plasticity of many of the stuccoes, the committee is convinced that plasticity is and can be easily obtained without sacrificing quality.

The committee has followed the established precedent of using compressive strength and absorption as measures of quality. While a direct relation cannot be established between the test results on cubes made and cured in the laboratory and stucco as applied to the wall, the committee believes that the test procedure outlined sets up a standard of quality that will insure reliable portland cement finish coat material.

The committee's study of finish coat stuccoes indicates that frequently poorly-graded aggregates are used, which necessitates the use of either very rich mixes or the addition of large amounts of plasticising agents. While the committee does not attempt to tell manufacturers how to make their stuccoes, it does feel that since it is possible through the use of very rich mixes to obtain the strength and absorption requirements proposed, this would be dangerous because it would tend toward crazing and shrinkage cracks. To guard against excess fines, the specification limits the amount of material passing a 100-mesh screen to 35 per cent of the total weight of the sample. This includes the cement and all other fine materials.

Your committee recommends that it be continued for the purpose of studying the reactions from this tentative specification, and also to revise the Standard Recommended Practice for Portland Cement Stucco, C-3a-23, to bring it up to date and make it of greater value to architects and specification writers.

This report has been submitted to letter ballot of the committee, which consists of 10 members, of whom 8 have voted affirmatively, none negatively, and 2 have not voted.

W. E. HART,
Chairman, Committee C-3

W. D. M. ALLAN,
Chairman, Sub-Committee 1

Discussion—Finish Coat Stucco Specifications

F. J. Spayth—The scratch coat and the brown coats of plaster are mixed approximately in the proportions of 3 parts sand to 1 part cement, or 2 parts sand and 1 part cement. This cement is not composed of 100 per cent cement, but is composed of portland cement and hydrated lime, the percentage of cement varying from 20 to 90. The minimum percentage is for interiors and the maximum percentage is for exteriors. **Mr. Spayth.**

Allow me to show you a test tube, which has a small piece of scratch coat in water. This scratch coat was on the wall for one year and this piece has been in water for over one year and is in good condition. The percentage of cement here was 25 by volume. This would be 40 per cent portland cement and 60 per cent finishing hydrated lime by weight.

Where the amount of cement in the base coat is equal to or greater than the amount of finishing hydrated lime, it is quite essential that the finishing hydrated lime be gauged with the same kind of cement.

Eighty-five per cent of the plaster in North America is finished with finishing hydrated lime from Ohio.

If we follow the recommendations of the committee, the percentage of cement will be increased from 100 to 400 per cent. That is 100 per cent in the base coat to 400 per cent in the finish.

It is not sound practice to increase the crystallization 400 per cent in ¼ of an inch. It is recommended that this increase be only 200 per cent. This will give a better bond and bring the coefficient of expansion and contraction more uniform between the coats.

When a contractor takes finishing hydrated lime at from one dollar to one dollar and fifty cents per 100 lb., instead of portland cement at 58 cents to 70 cents per 100 lb., it is to get the better project at more cost to the contractor.

Effect of Temperature on the Curing of Concrete

By R. A. Foley*

Concrete of various classifications is being used more and more in all seasons, irrespective of temperature or climatic conditions. It is of special economic importance to manufacturers of concrete products as well as to the contractors and engineers responsible for the design and construction of concrete structures to be properly informed as to the effect of temperature on the strength of concrete.

There have been many valuable and interesting papers and reports prepared, also a number of investigations made on this subject. The vast majority of this available information, however, deals almost entirely with so-called mass concrete. We have been able to discover but a very limited amount of available data and information on the subject applying directly to the manufacture of concrete products. It is, therefore, the purpose of this paper to outline briefly some practical information which we have been able to secure from actual practice in the manufacture of concrete products in a locality where seasonal, climatic and temperature conditions definitely prevail.

The ultimate result from a concrete mixture can be and is directly and definitely affected by several controlling factors, none of which can be slighted. We enumerate them as follows: (1) Proper balance of aggregate in accordance with fineness modulus theory, (2) proper quality of aggregate, (3) cement, (4) water-cement ratio, and (5) proper curing of finished product. The reason for placing the factor of curing as the fifth factor of importance is that the best results can be obtained from proper curing only after the other factors have been carefully adhered to.

At the time we entered into the manufacture of concrete pipe, about five years ago, we fortunately realized quite promptly the importance not only of producing a quality product but also of maintaining quality with a definite degree of uniformity. To be sure, many difficulties were encountered and experienced and we soon recognized the necessity of installing testing equipment and other apparatus which would serve accurately to determine the basic reasons for such irregularities. Since that time many thousands of dollars have been spent in testing, research and the like, and we attribute the steadily increasing measure of success which we have enjoyed to this fact. Guess work, inaccuracy and gross carelessness are bandits which have long since and still are preying upon the concrete products industry.

* Manager, Superior Products Co., Detroit, Mich.

It may be of interest to note some of the equipment which we consider necessary properly to control quality. Every concrete products plant should own or have access to a testing machine. In localities where there are several small plants, each plant could subscribe toward the maintenance of a testing laboratory. It would not be long in such cases before lively competition would be established through each endeavoring to outdo his neighbor in quality; in other words, the trilogy, quality, service and price would then be a reality.

Two testing machines are installed in our plants, a 20,000-lb. hand-operated hydraulic machine and a 100,000 lb. motor-driven machine. Both of these machines were purchased from a well-known manufacturer of testing equipment and their cost was by no means excessive. They have paid dividends continually since the day they were installed. Our curing kilns average in size 20 x 100 ft. and are equipped for both direct steam and water curing. In order to insure uniform temperature control during the curing period we installed recording thermometers. The charts from these thermometers answer a lot of questions. In our small plant laboratories we have the necessary equipment for checking fineness modulus of aggregate and absorption and for making hydrostatic tests, colorometric tests, etc. Our cement is tested by local laboratories. As stated before, proper curing is but one factor in the chain and it is of extreme importance that this fact is recognized and proper attention given to the others.

In addition to proper equipment, and just as essential, if not more so, is the plant engineer. To prove further the value we place in tests and research, during the year just passed over 1400 complete tests and analyses were made either personally or under the direction of our plant engineer. A large number of tests were run by outside engineers on our products in our laboratories as well as by outside laboratories. The extent of all these tests can probably be better appreciated in the fact that approximately 12,000 ft. of concrete pipe alone was consumed.

Just recently we received an interesting letter from a friend who is engaged in the concrete products and pipe business. I wish to quote a paragraph from the letter: "I have an option to purchase all the pipe machines and equipment in plant X. The machines were purchased in 1925 and used about 1½ years and are in first class mechanical condition. The plant endeavored during its short life to manufacture concrete pipe in accordance with A.S.T.M. specifications without a testing machine; you can imagine the results."

Luck is a hidden asset, although it is quite apparent there are still a number of concrete products manufacturers who consider it a major asset. Successful manufacturers, regardless of the products being produced, are those who definitely control quality. Recently we noted with a great deal of satisfaction and interest this statement in our advertisement "Improved Quality Will Definitely Increase the Use of Concrete Pipe." Inasmuch as we are in agreement that quality control is very essential, let us now consider briefly one of the important factors affect-

ing one way or the other the quality of concrete products, namely, temperature.

All consistencies of concrete show proportionally large increases in strength under proper curing conditions. The so-called dry and semi-dry mixtures of concrete used principally in the manufacture of machine-made concrete products such as concrete pipe, building block and the like, show a proportionally greater increase in strength under normal temperatures and curing conditions than mixtures with a high water-cement ratio. This is due largely to difference in periods of hydration. Either extreme of temperature impairs the quality of concrete in practically the same proportion. This is more definitely true with wet mixtures than it is with dry. (The effect of humidity upon the initial curing of concrete is of very little consequence, with the possible exception of very high humidity, which tends to retard somewhat the initial set.) Variation in temperature, on the other hand, has a remarkable effect upon the setting time of cement. The tensile strength of concrete is affected by extremes in temperature to a greater degree than is compressive strength.

The rate of increase in strength decreases with the age of the specimen and this rate of increase is less at lower temperature than at high temperatures. Concrete, under uniform normal hardening temperatures (60 to 70 deg. F.) will show an increase in compressive strength at 7, 14 and 28 days of approximately 50, 75 and 90 per cent. Under temperatures from 20 to 40 deg. F. these percentages will average about 30 per cent lower, and under higher temperatures, of 80 to 110 deg. F., they will be at least 50 per cent higher, particularly during the period of from 2 to 14 days. We have found from actual practice, and proven by hundreds of tests, that a mechanically compacted semi-dry concrete mass having a low water-cement ratio will attain prescribed 28-day compressive strengths within a period from 2 to 7 days if the mass is cured in a saturated atmosphere having a temperature of from 90 to 110 deg. F. during the initial period of curing.

In connection with all manufacturing operations economic limits must prevail to a greater degree than theoretical in order to insure economies and quality within safe and fair limits. After considerable investigation, we have placed time limits on curing of our products at a minimum of 48 hr. during the winter months. A longer initial curing period in freezing weather better secures the product against temperature shock. This is more particularly true from our observation as far as the manufacture of concrete pipe is concerned. Concrete block and like products are affected to a lesser degree when stored in the open after the initial curing in kilns than is concrete pipe, which has a thin wall structure and a large surface area exposed. An interesting fact which we have discovered is that plain concrete pipe, after having been cured in kilns for 48 hr. under temperatures of from 90 to 110 degs. F., when stored in the open in freezing temperatures shows a decided recession in strength during the period from 3 to 10 days. Specimens tested have shown this recession in strength at times to be as high as 25 per cent. We have only been able to explain

this condition by saying it is due to temperature shock as in the vast majority of cases the specimens tested have regained their early strengths during the period of from 14 to 21 days and beyond this age up to 60 or 90 days slowly but definitely increase in strength. Observations have been made during the severe winter months under almost every combination of temperature and atmospheric conditions. For example, specimens have been tested at an age of 7 days after having been cured for periods of from 48 to 60 hr. in temperatures of 100 deg. F. and then stored in the open in freezing temperatures, and almost invariably under such conditions a decrease in compressive strength is noted compared with 3-day tests. Again specimens from the same lot have been tested at 9 to 10 days after 24 or 48 hr. of clear weather with temperatures 15 or 20 deg. higher than the average temperatures during the 7-day period and

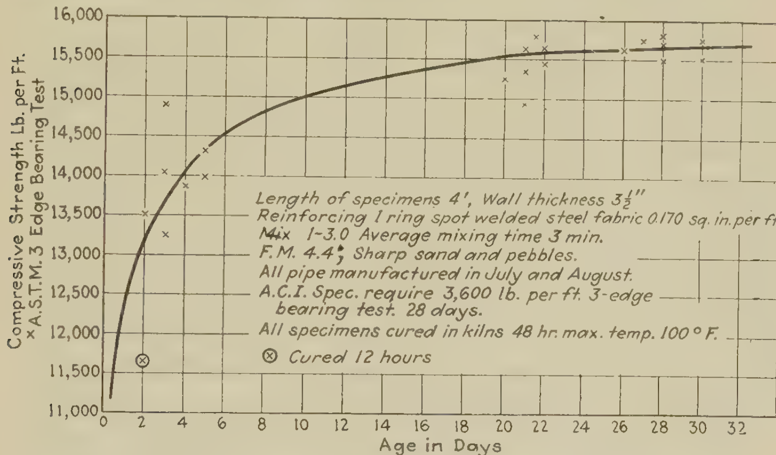


FIG. 1.—COMPRESSION STRENGTH CURVE FOR 18 SPECIMENS OF 30-IN. REINFORCED CONCRETE PIPE MANUFACTURED IN JULY AND AUGUST.

the strength tests have proven to be normal. In other words, under such conditions, temperature changes have had a very decided effect. On the other hand, specimens have been tested at the end of the initial curing period (48 to 60 hr.) showing a normal 28-day strength. Identical specimens were then stored in the open in freezing temperatures showing a recession in strength after testing at 5 to 7 days and additional tests at 28 days, and in a few cases 60 days, proved the product had not returned to the original 3-day strengths. This is a very unusual condition and was experienced during the past winter, 1927. We frankly admit we were unable to definitely discover the basic reasons. It is interesting to note that the product thus affected gradually returned to normal strength with increase in temperatures during the spring months. These particular tests are mentioned herewith as a matter of interest also on account of the fact that we believe them to be quite unusual. As a matter of fact,

this is the most extreme condition, as far as the effect of temperatures is concerned, which we have experienced since entering into the manufacture of concrete products.

Referring again to the controlling factors which definitely affect the quality of concrete, we strive at all times to maintain uniform conditions. For example, the maximum kiln temperature does not vary more than 10 deg. over the entire year. The water-cement ratio is held constant regardless of the season of the year. The fineness modulus tests as well as the quality of the aggregate used is uniformly maintained within practical limits. The mixing time is held uniform and of course the importance of the quality of cement is not neglected. If the foregoing important factors are strictly adhered to and uniformly maintained, temperature, humidity and precipitation are on the other hand the important factors

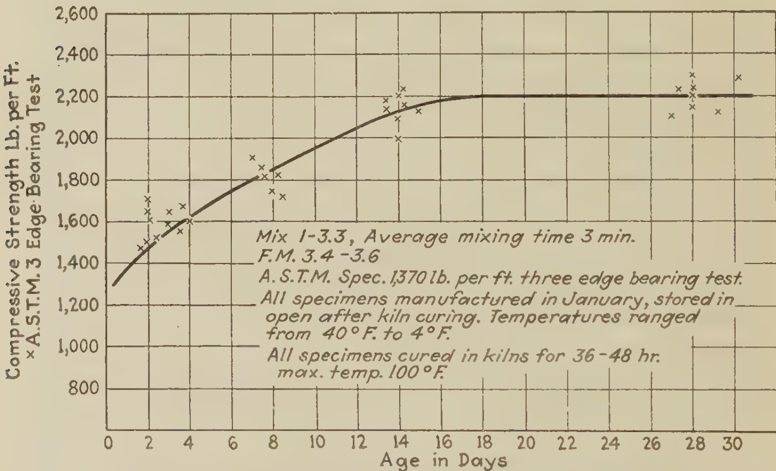


FIG. 2.—COMPRESSION STRENGTH CURVE FOR 33 SPECIMENS OF 15-IN. PLAIN CONCRETE PIPE MANUFACTURED IN JANUARY.

affecting one way or another the curing of the finished product in the open. In general, the warm, humid and rainy seasons are preferred to secure the highest strength.

The curves herewith offer additional proof of the value of maintaining higher temperatures during the initial period of curing, regardless of the season of the year. The curve in Fig. 1 was developed from a series of tests for compressive strength on 30-in. reinforced concrete pipe. All of the specimens were manufactured during the months of July and August. Specification strengths were secured in from 2 to 7 days; in fact, the increase in strength after the 7-day period was very gradual. The curve in Fig. 2 was developed from compressive strength tests on specimens of 15-in. plain concrete sewer pipe. All of the specimens tested were manufactured during the month of January and were stored in the

open after the initial period of curing in temperatures ranging from 30 to 40 deg. F. It will be noted also from these curves that the same high early strengths were secured during the period from 2 to 7 days as are shown in Fig. 1. The curve in Fig. 3 shows very definitely the effect of temperature during the curing period on average 1:2:4 concrete mix-

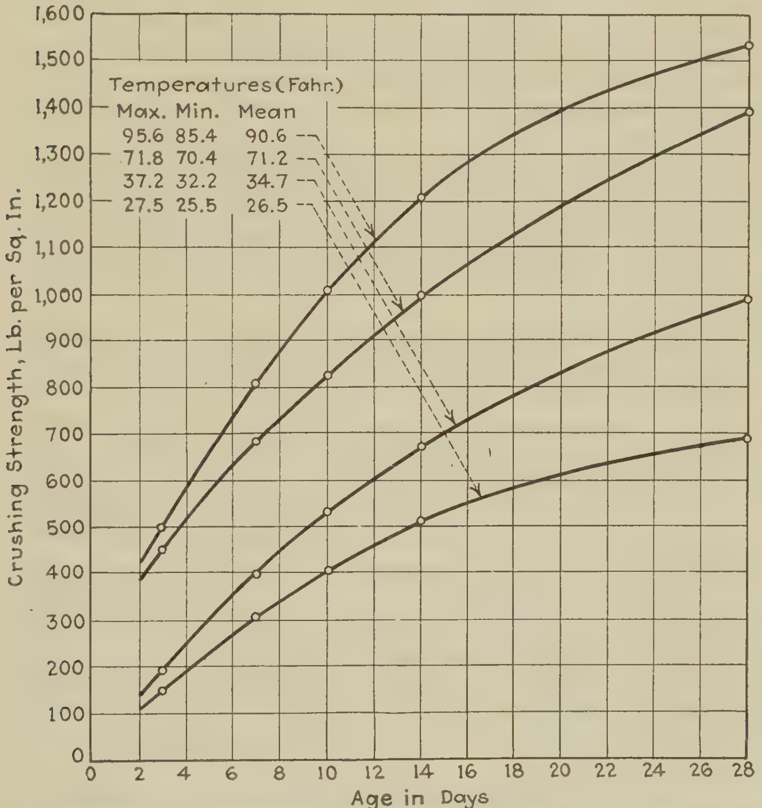


FIG. 3.—STRENGTH CURVES SHOWING EFFECT OF CURING TEMPERATURES ON 1:2:4 CONCRETE.

tures. This curve was secured from Bulletin No. 81, University of Illinois, Influence of Temperature on the Strength of Concrete, by A. B. McDaniel.

Very few existing standard specifications make any reference to temperature control of test specimens. The Joint Concrete Culvert Pipe Committee Specifications prohibit the testing of concrete pipe when they are frozen or when they are at a temperature below 40 deg. F. Here again the type of concrete evidently has an important bearing as we have tested specimens at temperatures below freezing which were just as satis-

factory as identical specimens which had been thoroughly thawed and dried before testing.

As stated before, very little data is available pertaining to the effect of the several important control factors on the quality of a mechanically compacted, semi-dry concrete mixture. This particular type of concrete cannot be directly compared in characteristics under various conditions with wet concrete mixtures. Our observations to date, which have been limited, nevertheless have proven the value and importance of further research and study concerning the direct effect of other important controlling factors on the quality of semi-dry mechanically compacted concrete mixtures.

In further reference to prevailing standard specifications for concrete pipe and products, it is interesting to note that very little attention has been given to the subject of proper curing. National associations, such as the American Concrete Institute, could well afford for the benefit of manufacturers particularly to sponsor further research and investigation on this subject.

Conclusions.

(1) The five important control factors affecting a concrete mixture must be uniformly maintained before the best results can be obtained through proper curing.

(2) A concrete mixture with a low water-cement ratio will show a proportionately higher increase in strength during the initial curing period than will a wet mixture.

(3) High temperatures during the initial curing period assists in producing high early strengths. The rate of increase in strength decreases with age of the product.

(4) Economic limits for the initial curing period are 48 hr. maximum during the summer months and 60 hr. maximum during the winter months with maximum temperatures of 90 to 110 deg. F.

(5) Plain concrete pipe and similar thin wall concrete products are more sensitive to temperature changes during the curing period than are mass products such as concrete block, brick, etc.

(6) Best results are obtained from outside curing during warm, humid and rainy seasons.

(7) Prevailing specifications should be more specific in reference to moisture and temperature control. Proper methods of curing concrete of all types should be more definitely prescribed in standard specifications.

(8) Further and more detailed study and research should be made, particularly in reference to the effect of the various control factors on semi-dry mechanically compacted concrete mixtures.

(9) These observations and conclusions have been made from practical application and experience in connection with the manufacture of concrete pipe, both plain and reinforced, and concrete building units and specialties.

Concrete Products Plant Operation

Report of Committee P-6

Your committee on Concrete Products Plant Operation conducted three series of tests to determine the effect of variations in mixing time on the strength of concrete block. Two series were conducted at the plant of the Consolidated Concrete Machinery Corporation, Adrian, Michigan, and one at the Arlington Concrete Products Company, Arlington Heights, Illinois. The results of these tests and two experiments conducted previously as a part of other research are given in the following report and three supplements.

This problem was chosen for investigation because of the conflicting opinions of products manufacturers on the value of long time mixing, and the best proportions of dry to wet mixing in plants using non-plastic mixes.

Ten years ago hand mixing was common. With the appearance of the small batch and continuous mixer, experience showed that even one or two minutes machine mixing produced better concrete than was obtained with hand mixing.

Data on plastic concrete also proved that an increase in strength and workability was possible with mixing times up to 5 minutes. This information gave rise to the present tendency to increase the total mixing time in products plants to 15 or even 20 minutes. The industry is now divided into two groups; one claiming that mixing 5 minutes is satisfactory, the other saying that as much as 20 minutes of mixing is necessary.

The Committee started to find the answer to this problem. When the first test results were available it was found that the increase in strength from the shortest to the longest mixing time either wet or dry or total was in many cases smaller than variations produced by differences in handling, curing and testing under normal products plant operation. For this reason the data presented in this report should not be taken as conclusive and the Committee recommends that if further tests are required that they be conducted in a laboratory where all variables can be rigidly controlled.

Of the 15 members of this Committee 13 voted affirmatively, none negatively and two refrained from voting on this report.

E. Grant Lantz,
Secretary.

RELATION OF MIXING TIME TO STRENGTH OF CONCRETE BLOCK

Submitted by Committee P-6

The purpose of the tests described in this report was to determine:

(1) Relation of wet mixing time to the strength of concrete block.
(2) The effect of variation in mixer blade type on the strength of concrete block.
(3) The mixing characteristics of 9 and 21-cu. ft. mixers as affecting the strength of concrete block.
(4) The mixing characteristic of a 21-cu. ft. mixer containing various volumes of combined aggregates as affecting the strength of concrete block.

The block were manufactured and cured at the plant of the Consolidated Concrete Machinery Corporation, Adrian, Mich., and were then moved to the laboratory of the Concrete Products Association of Detroit for testing. Acknowledgment is made to the Portland Cement Association, the Consolidated Concrete Machinery Corporation and the Concrete Products Association of Detroit for the generous cooperation afforded the Committee in carrying out these tests.

OUTLINE OF TESTS

The tests were divided into two series, the first using a 1:6 mix and the second using a 1:7 mix, both dry and rodded volumes. For convenience the first was called Series 6 and the second Series 7. Aggregate for both series was sand and gravel.

Grading of Aggregate—A constant grading for combined aggregate of 3.25 fineness modulus was maintained for both Series 6 and 7. Sieve analyses for the aggregate are given in Table No. 1.

Quantity of Cement—For Series 6 a 1:6 mix was used and for Series 7 a 1:7 mix.

Curing—Units for both Series 6 and 7 were stored in a closed building and sprinkled twice each day for the first 3 days of the curing period, and then allowed to dry in either the air of the storage shed or the testing laboratory. For Series 6, block tested at 10 days were shipped to the laboratory October 13 and for the 28-day test on October 22. For Series 7, the block tested at 7 days were shipped July 10 and for the 28-day test July 27.

Time of Mixing—For Series 6 total mixing time ranged from two to 16 minutes and for Series 7 from one to 15 minutes. The distribution of the total time between wet and dry mixing is given in Tables No. 2 and No. 6. The curves for Series 7 are plotted on the basis of wet mixing time only.

Ratio of Volume of Combined Aggregate to Mixer Capacity—For Series 7, a 21-cu. ft. mixer was operated with 14, 10.5 and 7 cu. ft. of combined aggregate, dry and rodded volumes.

Amount of Mixing Water—Trial batches were run to find the least amount of mixing water which could be used and still produce a unit with the minimum of web cracking with a total mixing time of 1 min. wet. For the mix and grading used and for both series a net water-cement ratio of 0.98, that is, 7.31 gals. per sack of cement was found to produce the desired results.

There may be some question as to why the same water-cement ratio was used with 1:6 and 1:7 mixes but as has been explained the amount of mixing water was decided upon by a visual inspection of the units produced with a total mixing time of 1 min. wet. The machine operator decided that the units of the 1:6 mix with water-cement ratio 0.98 and mixed a total of 1 min. each were about as dry as the machine could handle. After a careful checking of the 1:7 mix, the operation of the machine and the amount of breakage, the same amount of water was used for both mixes.

Tamping—The machine was equipped with an automatic device which gave each unit 9 full strokes of each tamper foot.

Age at Test—Blocks were tested for compressive strength at 10 and 28 days for Series 6 and at 7 and 28 days for Series 7.

MANUFACTURE OF BLOCK

The specimens for Series 6 were made October 9, 10 and 11, the specimens for Series 7, July 5, 6 and 7.

The sand and pebbles were stored in piles under a small wooden roof that gave some protection from the sun and rain.

A storage building was converted into a manufacturing plant for the test. The tamper was securely fastened to a concrete floor and the mixers set on a wooden platform about 3 ft. high.

All of the aggregate was measured in 1-cu. ft. boxes. The mixing water was weighed and corrections made for variations in the water content of the aggregate.

Both the 9 and 21-cu. ft. mixers were of the batch type. All tests in the 9-cu. ft. mixer were with spiral blades. A change from spiral to shovel blades was made with the 21-cu. ft. mixer to study the difference in mixing action. All batches were completely used before the next batch was dumped. As an additional precaution the first three units from each batch were discarded.

The block machine was of the stripper type with 4 full-width mechanical tamper feet, making a standard 3-core 8 x 8 x 16-in. unit. Concrete was fed into the machine hopper by a drag elevator and into the mold box by an automatic feeder. The machine was handled by an experienced operator.

Block were placed on steel racks. All of the units were sprinkled with a hose twice daily during the first 3 days. Remainder of curing took place in air under cover.

Units for test were selected from single batches.

Testing

Blocks were tested for compressive strength by the method prescribed in the American Concrete Institute Standard Specifications for Concrete Building Block and Tile except that they were not dried before testing.

Discussion of Tests

Tables No. 3 to No. 5 inclusive show in detail the compressive strengths for units manufactured under the conditions of Series 6; results for Series 7 are shown in Tables No. 7 to No. 11 inclusive.

(1) *Relation of wet mixing time to the strength of concrete block*—The results of these tests are based on units from only one batch so that no check is provided on the points not lying on the curves presented.

From the data presented it appears that with the exception of the 9-cu. ft. mixer in Series 6 that there is little strength to be gained by mixing longer than 5 or possibly 7 min. after the water is added.

The apparent reduction in strength of the batches run through the 21-cu. ft. mixer at full capacity may in part be explained by the fact that as the wet mixing time increased the mix became more plastic. The mix which was as dry as the machine could handle with 1 min. wet mixing was as plastic as the machine could handle with 3 min. dry and 12 min. wet mixing.

The reduction in strength may have been due to the difference in compaction of the tamping feet acting on a semi-plastic mix instead of a non-plastic mix. It was noticed that there was a visible slump in a number of block. There may have been enough slump in the units tested, even though not apparent with visual inspection, to produce an eccentric loading which would decrease the compressive strength of the block.

(2) *The effect of variation in blade type on the strength of concrete block*—The results of Series 6 show that there is a slight advantage in spiral blades up to 7½ min. wet mixing time. In Series 7 there appears to be a slight advantage with shovel blades. The difference in strength is, however, not great enough to establish one type of blade as being more desirable in batch mixers than the other. Economy in operation and maintenance is probably more important in determining the type to be used.

(3) *The mixing characteristics of 9 and 21-cu. ft. mixers as affecting the strength of concrete block*—In both Series 6 and 7 the curves tend to rise for the longer mixing periods, 12 to 15 min., indicating that increased strengths may possibly be obtained with long time mixing in 9-cu. ft. mixers.

The results for the 21-cu. ft. mixers show that there is no gain in strength with 12 to 15 min. wet mixing over 5 or possibly 7 min. wet mixing for a constant water-cement ratio. The results might have been different if consistency instead of water-cement ratio had been a constant.

(4) *The mixing characteristics of 21-cu. ft. mixers running at less than full capacity*—For Series 7 a 21-cu. ft. mixer containing 14 cu. ft. of mixed aggregate dry and rodded produced a block with a strength of approximately 1200 lb. per sq. in. of gross area.

In Series 7 a 21-cu. ft. mixer containing 7 cu. ft. of combined aggregate dry and rodded produced a mix of practically constant strength for a wet mixing time up to and including 12 mins. This strength was a few pounds above that for a 21-cu. ft. mixer containing 14 cu. ft. of aggregate dry and rodded even though the blades were in contact with the mix for a shorter part of each revolution and there was little noticeable movement of the mix back and forth across the length of the mixer.

With this mixer containing 10.5 cu. ft. of mixed aggregate, dry and rodded, there was sufficient mixing action to produce results similar to those for the same mixer containing 14 cu. ft. of combined aggregate, dry and rodded.

TABLE 1—SIEVE ANALYSIS OF AGGREGATE.

Per cent coarser than a given sieve.

Sieve Size or Number	Sand Series 6	Gravel Series 6	Sand Series 7	Gravel Series 7	Grading Combined Aggregate	
					Series 6	Series 7
100	98.4	99.6	97.8	99.0	98.5	97.9
48	85.2	98.9	82.8	96.9	86.5	84.6
28	54.7	97.5	50.1	94.2	59.0	56.3
14	34.1	96.0	32.3	93.2	40.3	40.8
8	19.3	91.3	18.0	91.5	26.5	28.3
4	6.8	72.0	5.5	82.2	13.3	16.2
3/8	....	13.1	....	15.5	1.3	2.2
Fineness modulus	2.98	5.68	2.86	5.72	3.25	3.26

TABLE 2—EFFECT OF VARIATION IN MIXING TIME.

Tests of 8 x 8 x 16-in. concrete building block.
Aggregate—Glacial sand (0-No. 4), gravel (No. 8 to ⅜-in.).
Fineness modulus of mixed aggregate = 3.25.
Mix 1:6 by volume dry and rodded aggregate.
Machine mixed concrete.
Number of tamps each of four feet—9.
Curing under cover by sprinking.
Water-cement ratio—0.98.
Each value the average of 5 tests except * the average of 4 tests.

[c] These units did not break at the capacity of the machine.

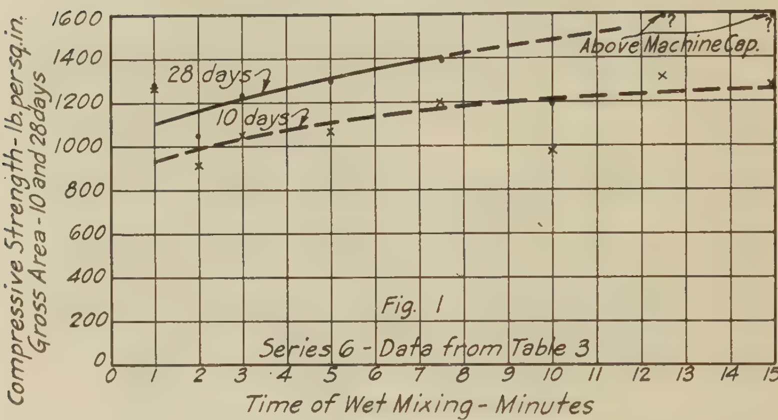
Ref.	Size of Mixer	Cu. Ft. of Mixed Aggregate Dry and Rodded Volume	Type of Mixing Blade	Time of Mixing, minutes		Compressive Strength, lb. per sq. in. Gross Area at 28 Days	Block per Sack Cement
				Dry	Wet		
6–1	9	6.0	Spiral	1	1	1279	16
6–2	9	6.0	"	1	2	1044	
6–3	9	6.0	"	1	3	1238*	
6–4	9	6.0	"	1	5	1297	
6–5	9	6.0	"	1	7½	1387	
6–6	9	6.0	"	1	10	1212	
6–7	9	6.0	"	1	12½	1587 [c]	
6–8	9	6.0	"	1	15	1587 [c]	
6–11	21	12.0	Spiral	1	1	1194	17
6–21	21	12.0	"	1	2	1431	
6–31	21	12.0	"	1	3	1402	
6–41	21	12.0	"	1	5	1319*	
6–51	21	12.0	"	1	7½	1499	
6–61	21	12.0	"	1	10	1448	
6–71	21	12.0	"	1	12½	1250	
6–81	21	12.0	"	1	15	1304	
6–12	21	12.0	Shovel	1	1	1078*	17
6–22	21	12.0	"	1	2	1203	
6–32	21	12.0	"	1	3	1502*	
6–42	21	12.0	"	1	5	1273	
6–52	21	12.0	"	1	7½	1414	
6–62	21	12.0	"	1	10	1150*	
6–72	21	12.0	"	1	12½	1414	
6–82	21	12.0	"	1	15	1351	

TABLE 3—DETAIL TEST RESULTS FOR 1:6 MIX.

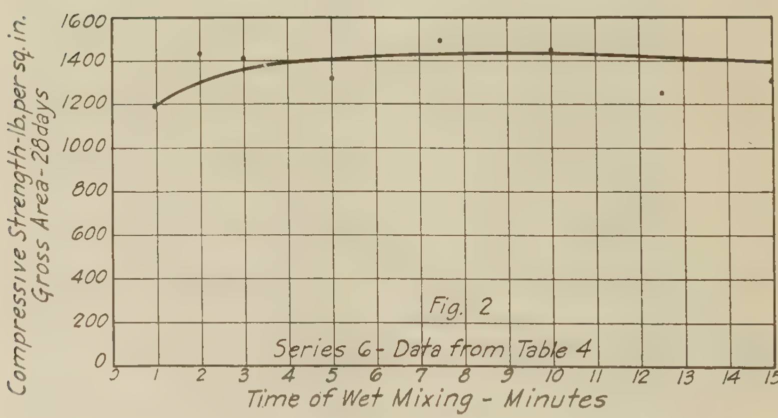
9-cu. ft. mixer equipped with spiral blades mixing 6 cu. ft. of mixed aggregate dry and rodded volume.

Batch	Time of Mixing, minutes		Compressive Strength of 8 x 8 x 16-in. Concrete Block, lb. per sq. in. Gross Cross-Sectional Area			
	Dry	Wet	10 Days	Average, 10 Days	28 Days	Average, 28 Days
6–1	1	1	1041		1057	
6–1			1131		1077	
6–1			1233		1431	
6–1			1302		1328	
6–1			1426	1227	1504	1279
6–2	1	2	1015		1293	
6–2			964		875	
6–2			872		875	
6–2			768		1151	
6–2			924	909	1024	1044
6–3	1	3	924		1176	
6–3			1022		1275	
6–3			1152		1194	
6–3			1132		1307	
6–3			1056	1057	....	1238
6–4	1	5	956		1184	
6–4			1136		1284	
6–4			1136		1284	
6–4			1008		1366	
6–4			1109	1069	1366	1297
6–5	1	7½	1200		1476	
6–5			1338		1360	
6–5			1136		1476	
6–5			1152		1284	
6–5			1140	1193	1338	1387
6–6	1	10	936		1131	
6–6			868		1275	
6–6			926		1284	
6–6			808		922	
6–6			832	874	1450	1212
6–7	1	12½	1373			
6–7			1365			
6–7			1229			
6–7			1269			
6–7			1269	1301	1587 [c]	1587
6–8	1	15	1541			
6–8			1186			
6–8			1194			
6–8			1277			
6–8			1194	1278	1587 [c]	1587 [c]

[c] These units did not break at the capacity of the machine.



Compressive Strength - lb. per sq. in. Gross Area - 10 and 28 days
1600
1400
1200
1000
800
600
400
200
0
28 days
10 days
Above Machine Cap.
Fig. 1
Series 6 - Data from Table 3
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15
Time of Wet Mixing - Minutes



Compressive Strength - lb. per sq. in. Gross Area - 28 days
1600
1400
1200
1000
800
600
400
200
0
Fig. 2
Series 6 - Data from Table 4
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15
Time of Wet Mixing - Minutes

TABLE 4—DETAIL TEST RESULTS FOR 1:6 MIX.

21-cu. ft. mixer equipped with spiral blades mixing 12 cu. ft. of mixed aggregate dry and rodded volume.

Batch	Time of Mixing, minutes		Compressive Strength of 8 x 8 x 16-in. Concrete Block, lb. per sq. in. Gross Cross-Sectional Area			
	Dry	Wet	10 Days	Average, 10 Days	28 Days	Average, 28 Days
6–11	1	1	864		1218	
6–11			922		1245	
6–11			972		1293	
6–11			1031		1293	
6–11			1162	990	924	1194
6–21	1	2	1088		1452	
6–21			1100		1532	
6–21			1192		1382	
6–21			1058		1345	
6–21			1100	1108	1442	1431
6–31	1	3	1266		1282	
6–31			1139		1311	
6–31			1256		1288	
6–31			1266		1587	
6–31			1309	1247	1541	1402
6–41	1	5	1179		1293	
6–41			1147		1352	
6–41			1096		1338	
6–41			1032		1292	
6–41			1053	1101	....	1319
6–51	1	7½	1188		1587	
6–51			1032		1488	
6–51			924		1488	
6–51			1233		1392	
6–51			1152	1106	1538	1499
6–61	1	10	1318		1538	
6–61			1142		1373	
6–61			854		1388	
6–61			1142		1352	
6–61			1233	1138	1587	1448
6–71	1	12½	1015		1293	
6–71			964		1332	
6–71			1015		1352	
6–71			1015		1178	
6–71			1008	1003	1093	1250
6–81	1	15	872		1352	
6–81			938		1293	
6–81			1022		1194	
6–81			938		1299	
6–81			938	942	1382	1304

TABLE 5—DETAIL TEST RESULTS FOR 1:6 MIX.

21-cu. ft. mixer equipped with shovel blades mixing 12 cu. ft. of mixed aggregate dry and rodded volume.

Batch	Time of Mixing, minutes		Compressive Strength of 8 x 8 x 16-in. Concrete Block, lb. per sq. in. Gross Cross-Sectional Area			
	Dry	Wet	10 Days	Average, 10 Days	28 Days	Average, 28 Days
6–12	1	1	784		1154	
6–12			860		940	
6–12			768		1148	
6–12			894		1070	
6–12			908	843	....	1078
6–22	1	2	995		970	
6–22			900		1275	
6–22			1026		1292	
6–22			944		1086	
6–22			1026	978	1392	1203
6–32	1	3	908		1438	
6–32			1101		1587	
6–32			1026		1538	
6–32			972		1444	
6–32			1026	1007	....	1502
6–42	1	5	964		1284	
6–42			1007		1284	
6–42			1101		1293	
6–42			964		1330	
6–42			964	1000	1176	1273
6–52	1	7½	1078		1384	
6–52			1084		1452	
6–52			1152		1392	
6–52			1026		1538	
6–52			1010	1070	1306	1414
6–62	1	10	1208		940	
6–62			1261		1046	
6–62			1224		1306	
6–62			1136		1306	
6–62			1237	1213	....	1150
6–72	1	12½	1315		1442	
6–72			1070		1436	
6–72			1000		1442	
6–72			934		1384	
6–72			1221	1108	1366	1414
6–82	1	15	1152		1452	
6–82			1034		1468	
6–82			1338		1284	
6–82			1365		1468	
6–82			1062	1190	1085	1351

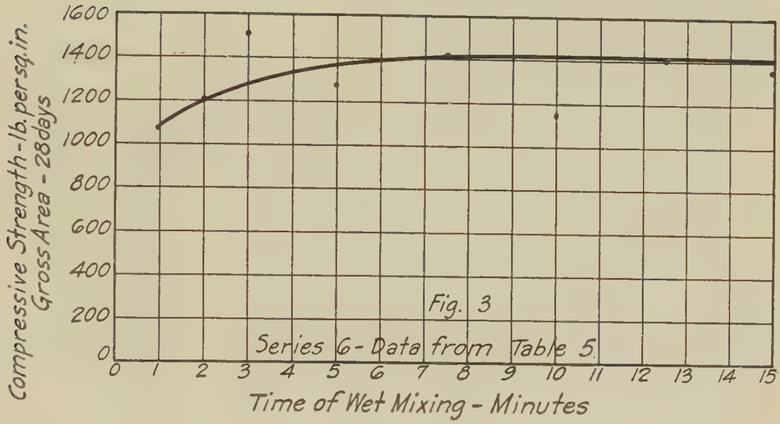


Fig. 3
Series 6 - Data from Table 5

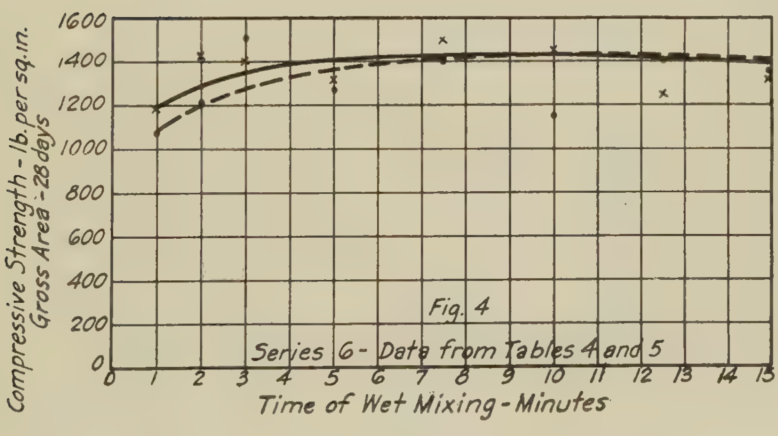


Fig. 4
Series 6 - Data from Tables 4 and 5

TABLE 6—EFFECT OF VARIATION IN MIXING TIME.

Tests of 8 x 8 x 16-in. concrete building block.
Aggregate—glacial sand (0–No. 4), gravel (No. 8 to ⅜-in.).
Fineness modulus of mixed aggregate = 3.25.
Mix 1:7 by volume dry and rodded aggregate.
Machine mixed concrete.
Number of tamps each of four feet—9.
Curing under cover by sprinkling.
Water-cement ratio—0.98.
Each value the average of 5 tests.

Ref.	Size of Mixer	Cu. Ft. of Mixed Aggregate Dry and Rodded Volume	Type of Mixing Blade	Time of Mixing, minutes		Compressive Strength, lb. per sq. in. Gross Area at 28 Days	Block per Sack Cement
				Dry	Wet		
7–1	9	7.0	Spiral	0	1	1105	20
7–2	9	7.0	"	0	2	1145	
7–3	9	7.0	"	1	1	1061	
7–4	9	7.0	"	0	3	1178	
7–5	9	7.0	"	1½	1½	975	
7–6	9	7.0	"	0	5	1153	
7–7	9	7.0	"	2½	2½	1172	
7–8	9	7.0	"	3	7	1207	
7–9	9	7.0	"	3	12	1443	
7–11	21	14.0	Spiral	0	1	1002	20
7–21	21	14.0	"	0	2	1215	
7–31	21	14.0	"	1	1	1198	
7–41	21	14.0	"	0	3	1176	
7–51	21	14.0	"	1½	1½	1150	
7–61	21	14.0	"	0	5	1122	
7–71	21	14.0	"	2½	2½	1149	
7–81	21	14.0	"	3	7	1242	
7–91	21	14.0	"	3	12	938	
7–12	21	14.0	Shovel	0	1	1183	20
7–22	21	14.0	"	0	2	1051	
7–32	21	14.0	"	1	1	1167	
7–42	21	14.0	"	0	3	1236	
7–52	21	14.0	"	1½	1½	1243	
7–62	21	14.0	"	0	5	1237	
7–72	21	14.0	"	2½	2½	1094	
7–82	21	14.0	"	3	7	1227	
7–92	21	14.0	"	3	12	1168	
7–13	21	10.5	Spiral	0	1	1081	20
7–73	21	10.5	"	2½	2½	1193	
7–83	21	10.5	"	3	7	1316	
7–93	21	10.5	"	3	12	994	
7–14	21	7.0	Spiral	0	1	1202	20
7–24	21	7.0	"	0	2	1256	
7–34	21	7.0	"	1	1	1268	
7–44	21	7.0	"	0	3	1213	
7–54	21	7.0	"	1½	1½	1216	
7–64	21	7.0	"	0	5	1081	
7–74	21	7.0	"	2½	2½	1211	
7–84	21	7.0	"	3	7	1157	
7–94	21	7.0	"	3	12	1216	

TABLE 7—DETAIL TEST RESULTS FOR 1:7 MIX.

9-cu. ft. mixer equipped with spiral blades mixing 7 cu. ft. of mixed aggregate dry and rodded volume.

Batch	Time of Mixing, minutes		Compressive Strength of 8 x 8 x 16-in. Concrete Block, lb. per sq. in. Gross Cross-Sectional Area			
	Dry	Wet	7 Days	Average, 7 Days	28 Days	Average, 28 Days
7–1	0	1	773		1054	
7–1			823		1292	
7–1			701		1138	
7–1			773		1000	
7–1			639	742	1041	1105
7–2	0	2	778		1208	
7–2			772		1187	
7–2			809		1100	
7–2			893		1128	
7–2			947	840	1100	1145
7–3	1	1	778		1048	
7–3			693		1038	
7–3			733		1125	
7–3			677		1016	
7–3			733	723	1077	1061
7–4	0	3	733		1162	
7–4			816		1016	
7–4			773		1302	
7–4			773		1100	
7–4			854	790	1309	1178
7–5	1½	1½	794		924	
7–5			754		1085	
7–5			678		1016	
7–5			684		809	
7–5			639	710	1040	975
7–6	0	5	917		1182	
7–6			863		1077	
7–6			917		1077	
7–6			902		1232	
7–6			854	891	1195	1153
7–7	2½	2½	924		1195	
7–7			778		1203	
7–7			809		1286	
7–7			885		1085	
7–7			829	845	1093	1172
7–8	3	7	809		1270	
7–8			893		1232	
7–8			902		1161	
7–8			794		1294	
7–8			885	857	1077	1207
7–9	3	12	854		1494	
7–9			1090		1501	
7–9			1122		1306	
7–9			1081		1477	
7–9			1130	1055	1438	1443

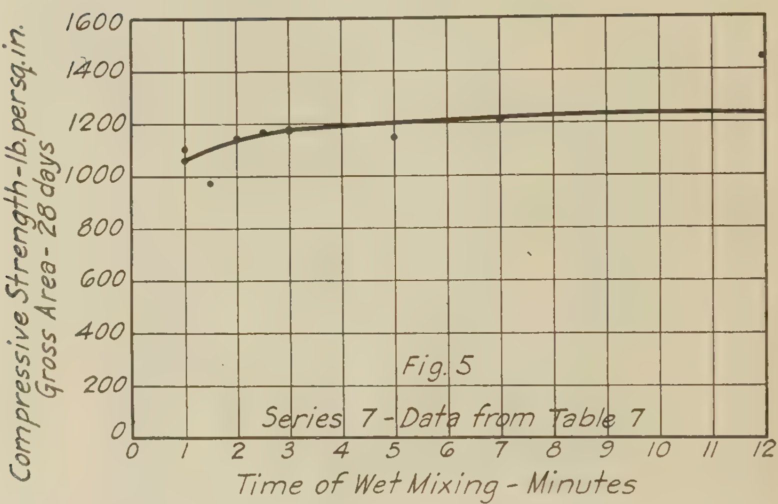


Fig. 5

Series 7 - Data from Table 7

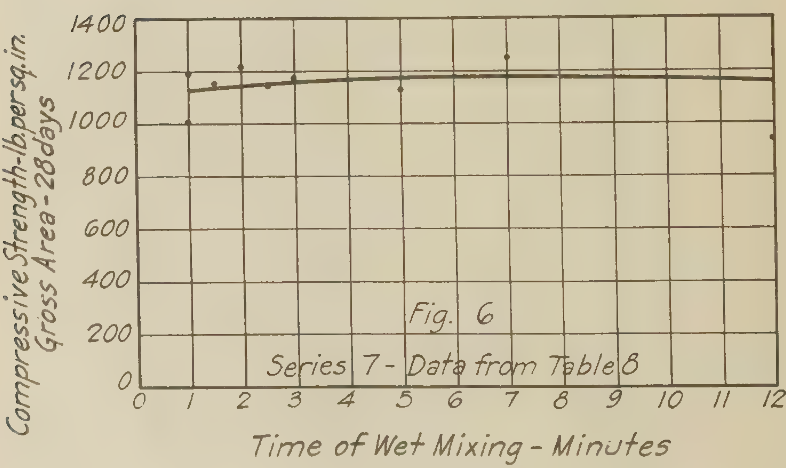


Fig. 6

Series 7 - Data from Table 8

TABLE 8—DETAIL TEST RESULTS FOR 1:7 MIX.

21-cu. ft. mixer equipped with spiral blades mixing 14 cu. ft. of mixed aggregate dry and rodded volume.

Batch	Time of Mixing, minutes		Compressive Strength of 8 x 8 x 16-in. Concrete Block, lb. per sq. in. Gross Cross-Sectional Area			
	Dry	Wet	7 Days	Average, 7 Days	28 Days	Average, 28 Days
7-11	0	1	964		964	
7-11			904		956	
7-11			808		1009	
7-11			668		1009	
7-11			786	826	1074	1002
7-21	0	2	904		1234	
7-21			904		1324	
7-21			809		1017	
7-21			904		1154	
7-21			977	900	1346	1215
7-31	1	1	832		1211	
7-31			754		1234	
7-31			858		1272	
7-31			816		1024	
7-31			917	835	1250	1198
7-41	0	3	977		1235	
7-41			1042		1108	
7-41			948		1061	
7-41			872		1151	
7-41			970	962	1324	1176
7-51	1½	1½	940		1063	
7-51			948		1000	
7-51			908		1152	
7-51			886		1334	
7-51			924	921	1200	1150
7-61	0	5	841		1078	
7-61			858		1078	
7-61			900		1184	
7-61			913		1122	
7-61			908	884	1150	1122
7-71	2½	2½	886		1131	
7-71			940		1101	
7-71			1011		1130	
7-71			1020		1233	
7-71			956	963	1150	1149
7-81	3	7	976		1150	
7-81			908		1354	
7-81			1002		1193	
7-81			1034		1253	
7-81			1077	999	1261	1242
7-91	3	12	807		813	
7-91			733		762	
7-91			754		956	
7-91			773		1111	
7-91			708	755	1048	938

TABLE 9—DETAIL TEST RESULTS FOR 1:7 MIX.

21-cu. ft. mixer equipped with shovel blades mixing 14 cu. ft. of mixed aggregate dry and rodded volume.

Batch	Time of Mixing, minutes		Compressive Strength of 8 x 8 x 16-in. Concrete Block, lb. per sq. in. Gross Cross-Sectional Area			
	Dry	Wet	7 Days	Average, 7 Days	28 Days	Average, 28 Days
7–12	0	1	715		1152	
7–12			756		1159	
7–12			724		1251	
7–12			733		1159	
7–12			800	746	1196	1183
7–22	0	2	617		977	
7–22			692		1071	
7–22			617		1023	
7–22			701		1079	
7–22			715	668	1103	1051
7–32	1	1	784		1145	
7–32			784		1233	
7–32			749		1161	
7–32			763		1167	
7–32			777	771	1129	1167
7–42	0	3	769		1161	
7–42			824		1188	
7–42			847		1387	
7–42			639		1275	
7–42			800	776	1169	1236
7–52	1½	1½	708		1038	
7–52			832		1283	
7–52			824		1315	
7–52			808		1259	
7–52			864	807	1323	1243
7–62	0	5	901		1259	
7–62			864		1338	
7–62			864		1283	
7–62			886		1119	
7–62			886	880	1188	1237
7–72	2½	2½	832		1119	
7–72			769		1137	
7–72			832		1079	
7–72			769		1015	
7–72			733	787	1119	1094
7–82	3	7	994		1241	
7–82			941		1137	
7–82			908		1153	
7–82			965		1346	
7–82			941	950	1259	1227
7–92	3	12	872		1198	
7–92			847		1127	
7–92			847		1315	
7–92			900		1038	
7–92			941	881	1161	1168

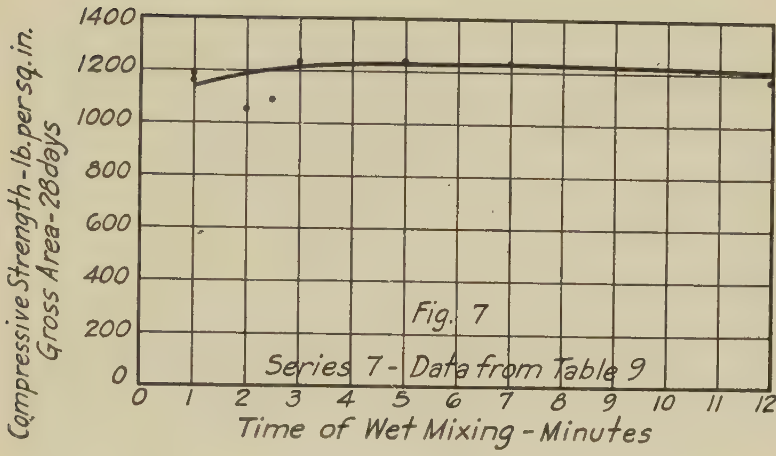


Fig. 7
Series 7 - Data from Table 9

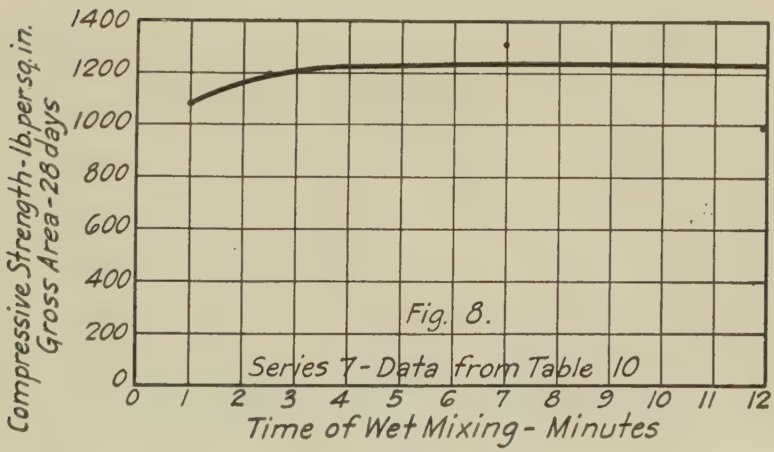


Fig. 8.
Series 7 - Data from Table 10

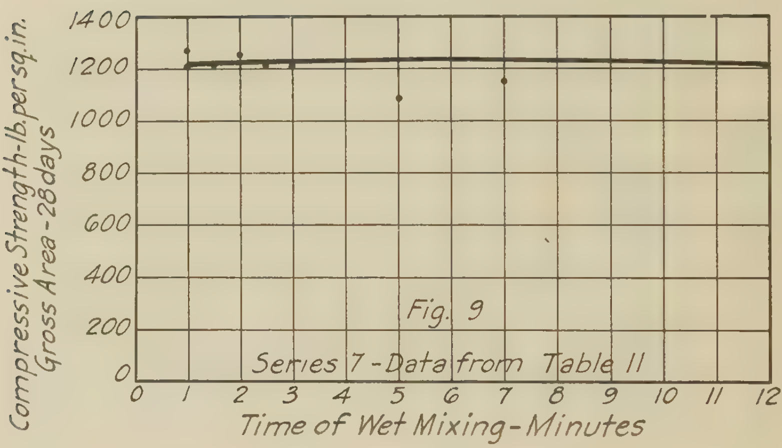


Fig. 9

Series 7 - Data from Table 11

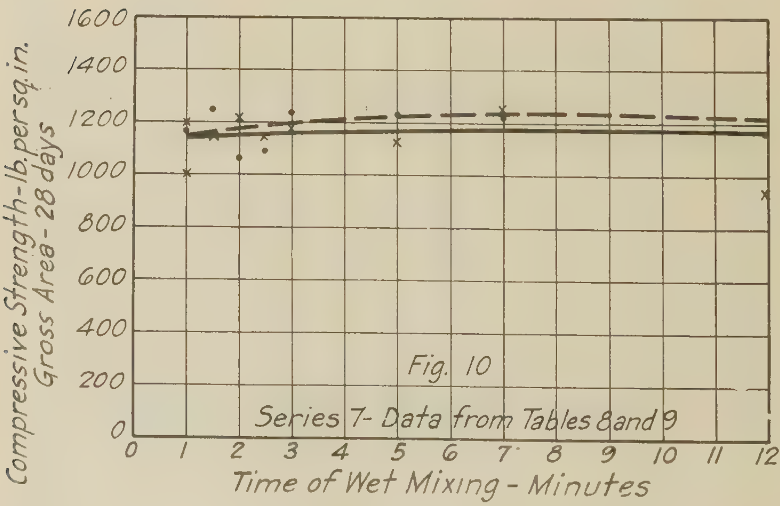


Fig. 10

Series 7 - Data from Tables 8 and 9

TABLE 10—DETAIL TEST RESULTS FOR 1:7 MIX.

21-cu. ft. mixer equipped with spiral blades mixing 10.5 cu. ft. of mixed aggregate dry and rodded volume.

Batch	Time of Mixing, minutes		Compressive Strength of 8 x 8 x 16-in. Concrete Block, lb. per sq. in. Gross Cross-Sectional Area			
	Dry	Wet	7 Days	Average, 7 Days	28 Days	Average, 28 Days
7–13	0	1	633		1103	
7–13			655		1145	
7–13			733		1125	
7–13			733		955	
7–13			677	686	1079	1081
7–73	2½	2½	893		1178	
7–73			816		1276	
7–73			816		1259	
7–73			824		1251	
7–73			886	847	1000	1193
7–83	3	7	847		1079	
7–83			908		1342	
7–83			839		1426	
7–83			816		1308	
7–83			824	847	1426	1316
7–93	3	12	740		1056	
7–93			701		977	
7–93			693		937	
7–93			715		1000	
7–93			624	695	1000	994

TABLE 11—DETAIL TEST RESULTS FOR 1:7 MIX.

21-cu. ft. mixer equipped with spiral blades mixing 7 cu. ft. of mixed aggregate dry and rodded volume.

Batch	Time of Mixing, minutes		Compressive Strength of 8 x 8 x 16-in. Concrete Block, lb. per sq. in. Gross Cross-Sectional Area			
	Dry	Wet	7 Days	Average, 7 Days	28 Days	Average, 28 Days
7–14..............	0	1	800		1262	
7–14..............			961		1115	
7–14..............			882		1233	
7–14..............			829		1071	
7–14..............			940	882	1330	1202
7–24..............	0	2	777		1358	
7–24..............			794		1307	
7–24..............			716		1179	
7–24..............			872		1288	
7–24..............			902	812	1150	1256
7–34..............	1	1	768		1262	
7–34..............			864		1216	
7–34..............			777		1150	
7–34..............			808		1395	
7–34..............			860	815	1316	1268
7–44..............	0	3	908		1251	
7–44..............			808		1270	
7–44..............			854		1150	
7–44..............			886		1162	
7–44..............			858	863	1233	1213
7–54..............	1½	1½	824		1157	
7–54..............			933		1130	
7–54..............			901		1195	
7–54..............			816		1387	
7–54..............			894	874	1210	1216
7–64..............	0	5	773		1117	
7–64..............			832		1095	
7–64..............			779		1208	
7–64..............			847		947	
7–64..............			770	800	1039	1081
7–74..............	2½	2½	723		1145	
7–74..............			808		1208	
7–74..............			902		1233	
7–74..............			872		1138	
7–74..............			841	829	1330	1211
7–84..............	3	7	769		1225	
7–84..............			723		1035	
7–84..............			756		1225	
7–84..............			693		1079	
7–84..............			740	736	1225	1157
7–94..............	3	3	777		1251	
7–94..............			847		1103	
7–94..............			924		1216	
7–94..............			994		1295	
7–94..............			948	898	1216	1216

TABLE 12—SIEVE ANALYSIS OF AGGREGATES.

Per cent coarser than a given sieve.

Sieve Size or Number	Sand-Gravel		Haydite	
	Sand	Gravel	Fine	Coarse
100	99.1	99.5	83.3	97.9
43	85.5	99.2	76.6	97.7
28	56.5	98.6	64.2	97.5
14	36.1	97.7	42.4	97.3
8	19.2	93.7	6.1	96.3
4	4.1	55.3	....	70.4
3/8	....	....	....	5.9
Fineness modulus	3.00	5.44	2.73	5.63

RELATION OF DRY MIXING TIME TO COMPRESSIVE STRENGTH OF CONCRETE BLOCK

Submitted by Committee P-6

The purpose of the tests described in this supplement was to determine the relation of dry mixing time to the compressive strength of concrete block and not to determine what total mixing time or combination of wet and dry mixing time is most economical in the manufacture of concrete masonry units.

The block were manufactured at the Arlington Concrete Products Company, Arlington Heights, Illinois. Acknowledgment is made to the Lehigh Portland Cement Company and to Messrs. Paul Taegge and Frank Busse of the Arlington Concrete Products Company for the generous cooperation afforded the Committee in carrying on these tests.

OUTLINE OF TESTS

The test was divided into 2 series, the first using sand-gravel as aggregate and the second using haydite as aggregate. For convenience the first was called Series 1 and the second Series 2.

Grading of Aggregate—The grading for each type of aggregate in both series was the same as used in normal plant operation. Sieve analyses for the aggregates are given in Table No. 12.

Quantity of Cement—At 1:6 mix, 4 parts sand and 2 parts gravel, damp and loose volume for sand-gravel aggregate was used as in regular plant operation. For haydite a 1:10 mix, 5 parts fine and 5 parts coarse, damp and loose was used as is also common practice in the Arlington plant.

Curing—The units from both series were placed on curing cars and stored in a heated room at about 60 deg. F. for 14 days and then removed to the laboratory where they were stored until tested.

Time of Mixing—For both series a constant wet mixing time of 5 min. was maintained with the exception of two groups, one mixed

1 min. dry and 9 min. wet, and one 3 min. dry and 7 min. wet. The curves for both series are plotted on the basis of dry mixing time only.

Ratio of Volume of Combined Aggregate to Mixer Capacity—For both series half-sack batches were used. For the sand-gravel series 3 cu. ft. of aggregate and for the haydite series 3 cu. ft. of aggregate, damp and loose volumes, were mixed in a 9-cu. ft. mixer.

Amount of Mixing Water—By running test batches it was found that 10 lb. of water added to the sand-gravel batches and 32 lb. for the haydite batches produced the consistency normally used.

Tamping—The machine was equipped with an automatic device that gave each unit 5 full strokes of each tamper bar.

Age at Test—All block were tested for compressive strength at 28 days.

MANUFACTURE OF BLOCK

All the units were made December 6, 1928, with no change in regular plant operation except that the aggregate and water were measured and controlled by the engineers in charge of the test.

The sand and gravel were stored outside the plant in unprotected piles. A layer about 6 in. thick was frozen over both piles, which protected the rest of the aggregate against freezing. The haydite was protected by a shed roof. None of the haydite contained enough free moisture to freeze into a covering or lumps.

All of the aggregate was measured in calibrated, straight sided pails holding 0.86 cu. ft. Mixing water and cement was weighed.

The mixer was charged with a skip loader. The skip was loaded with half the aggregate, then the cement and finally the rest of the aggregate. The mixer was of the batch type, equipped with spiral blades, and of 9 cu. ft. capacity. All batches were completely used before the next batch was dumped. As an additional precaution the first 3 units from each batch was discarded. When changing from sand-gravel to haydite an entire half-sack batch was run through the machine.

The block machine was of the stripper type, making standard 3-core 8 x 8 x 16-in. units. The concrete was dumped directly from the mixer into the machine hopper, and fed into the mold box by a power feeder, The machine was operated by the regular operator.

Blocks were placed on steel curing cars and pushed into the curing chamber which was heated with a warm air furnace. The units were not sprinkled. This type of curing contained for 14 days, remainder of curing in air of the laboratory.

Units for test were taken from two batches.

TESTING

Blocks were tested for compressive strength by the method prescribed in the American Concrete Institute Standard Specifications for Concrete Block except that they were not dried before testing.

DISCUSSION OF TESTS

Table No. 13 shows the results for sand-gravel series; results for the haydite series are shown in Table 14.

From the data presented it appears that for sand-gravel aggregate, the time of dry mixing does not affect the strength so long as there is sufficient total mixing time. Practically the same strengths were obtained with 1 min. dry and 9 min. wet mixing and 5 min. dry and 5 min. wet mixing. The word "dry" must be qualified for the sand contained 2.58 per cent moisture and the gravel 2.95 per cent. It is probable that the greater the amount of free moisture in the aggregate the greater the effect of mixing before the water is added.

For haydite there was an increase in strength for up to 10 min. of dry mixing, which may also have been due to the increase in total mixing time. The increase in strength from 5 to 10 min. dry mixing was only 98 lb. per sq. in. This increase might have been even greater if more of the total mixing time had been devoted to wet mixing. The results for no dry and 5 min. mixing after the water was added is out-of-line. The fine haydite contained 7.2 per cent moisture and the coarse 10 per cent. This is less moisture than will be absorbed by the aggregate in a 3-hr. period. The increase in strength is probably due to the longer mixing times providing a more thorough cement coating for the particles of aggregate so that when the water was added and the block tamped the more uniform coating of cement paste produced a unit of greater strength.

TABLE 13—EFFECT OF VARIATION IN DRY MIXING TIME ON THE COMPRESSIVE STRENGTH OF CONCRETE BLOCK.

Tests of 8 x 8 x 16-in. concrete building block.
Aggregate—Elgin sand (0–No. 4), gravel (No. 8—No. 4).
Fineness modulus of mixed aggregate = 3.88.
Mix 1:6 by volume damp and loose.
Machine mixed concrete.
Number of tamps—5.
Curing under cover—no sprinkling.
Water-cement ratio—0.71.
Each value the average of 6 tests, 3 from each of 2 batches except* the average of 5 tests, 3 from one batch and 2 from the other.

Ref.	Size of Mixer	Cu. Ft. of Aggregate, Damp and Loose	Type of Mixing Blade	Time of Mixing, minutes		Compressive Strength, lb. per sq. in. Gross Area at 28 Days	Block per Sack Cement
				Dry	Wet		
1–AB1......	9	3	Spiral	0	5	2003*	14
1–AB2......	9	3	"	1	5	2082	
1–AB3......	9	3	"	3	5	2224	
1–AB4......	9	3	"	5	5	2312	
1–AB5......	9	3	"	10	5	2278	
1–X21......	9	3	"	1	9	2478	
1–X22......	9	3	"	3	7	2203	

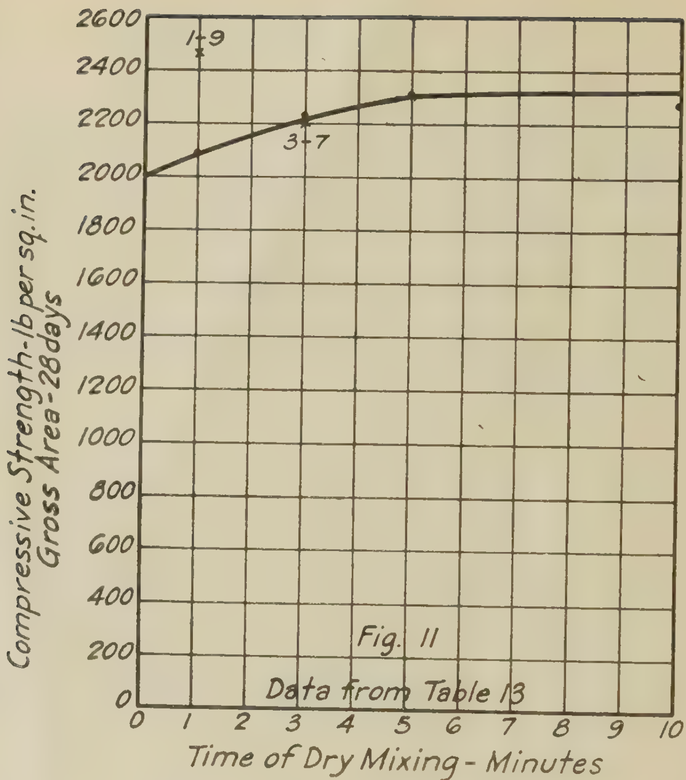


Fig. 11

Data from Table 13

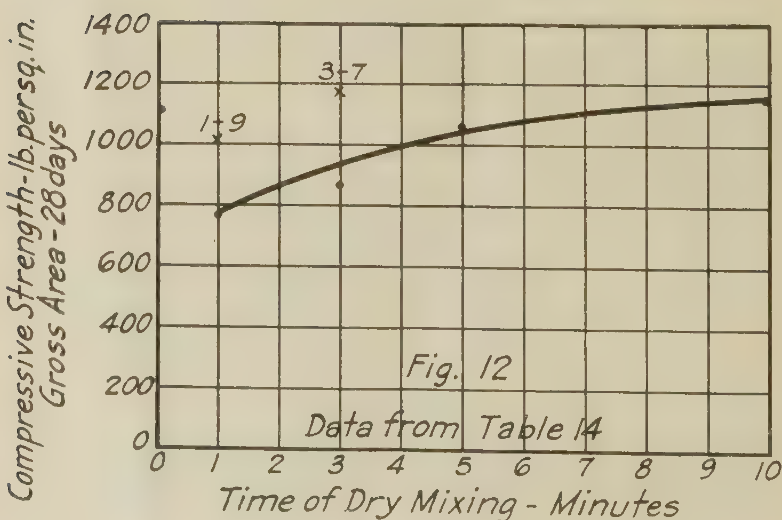


Fig. 12

Data from Table 14

As a matter of interest a 1 min. dry and 9 wet and 3 dry and 7 min. wet mix was used for both aggregates. The results are not conclusive but show that there may be a greater need for dry mixing with haydite than for sand-gravel aggregate.

TABLE 14—EFFECT OF VARIATION IN DRY MIXING TIME ON THE COMPRESSIVE STRENGTH OF CONCRETE BLOCK.

Tests of 8 x 8 x 16-in. concrete building block.
Aggregate—Danville Haydite, fine (0–No. 8), coarse (No. 8–3/8).
Fineness modulus of mixed aggregate = 4.18.
Mix 1:10 by volume damp and loose.
Machine mixed concrete.
Number of tamps—5.
Curing under cover—no sprinkling.
Each value the average of 6 tests, 3 from each of two batches.

Ref.	Size of Mixer	Cu. Ft. of Aggregate, Damp and Loose	Type of Mixing Blade	Time of Mixing, minutes		Compressive Strength, lb. per sq. in. Gross Area at 28 Days	Block per Sack Cement
				Dry	Wet		
2–AB1......	9	5	Spiral	0	5	1110	22
2–AB2......	9	5	"	1	5	767	
2–AB3......	9	5	"	3	5	869	
2–AB4......	9	5	"	5	5	1054	
2–AB5......	9	5	"	10	5	1152	
1–X21......	9	5	"	1	9	1010	
1–X22......	9	5	"	3	7	1182	

TIME OF MIXING TEST CONDUCTED AT PITTSBURGH, PENNSYLVANIA

Submitted by Committee P-6

This supplement presents the known facts covering the time of mixing test which was one of a series of experiments on methods of manufacturing concrete block conducted at the Iron City Brick and Stone Company during October and November, 1924. The units were tested at the Pittsburgh Testing Laboratory.

The specimens were made under regular plant operating conditions, the only departure being that the materials were proportioned and the consistency controlled by the engineers in charge. The mixer was of the batch type with revolving paddles. The block machine was of the stripper type with mechanical tampers making standard 8 x 8 x 16-in. hollow block. The temperature of the curing room was about 90 deg. F.

The following paragraph discussing the results of this test is quoted from the original report written by the engineer in charge of the tests.

"Thorough mixing adds greatly to the workability and strength of concrete. The results obtained from this series of tests on time of mixing dry concrete are shown in Fig. 13. Surprising increases in strength are

obtained by mixing the concrete up to at least two minutes. The Pittsburgh series might have shown comparatively greater strengths when the time of mixing was over 2 min. if more tests had been made. One minute mixing produced block having only 911 lb. strength per sq. in. (gross section) while under the same conditions 2 min. mixing produced block of 1243 lb. strength. (See Table 15.) The advantages gained through careful attention to other details of manufacture are largely lost if insufficient mixing be given the concrete. In the present series, a gain of 30 per cent in strength is noted by increasing the time of mixing from one to two minutes after the water was added."

TABLE 15—EFFECT OF TIME OF MIXING ON COMPRESSIVE STRENGTH OF CONCRETE BLOCK.

Compression tests of 8 x 8 x 16-in. concrete building block.
Aggregate, Ohio River sand 0 to $\frac{3}{16}$ in. and Ohio River pebbles $\frac{1}{4}$ to $\frac{3}{4}$ in.
Fineness modulus of mixed aggregate = 4.00.
Mix 1:5 by volume of dry rodded aggregate.
Machine mixed concrete.
All batches mixed dry while mixer blades made 4 revolutions and for length of time shown after the water was added.
Curing, 15 hr. in steam, remainder until test in air.
Age at test, 28 days.
Blocks per sack of cement about 14.
Each value is the average of 4 specimens, 2 each from 2 batches made on different days.

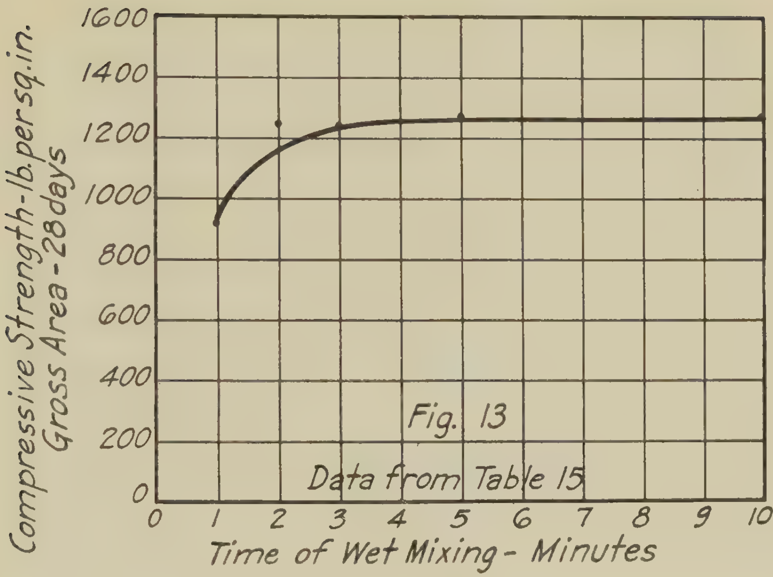
Time of Mixing, minutes	Compressive Strength, lb. per sq. in. (Gross Arae)
1	911
2	1243
3	1238
5	1263
10	1265

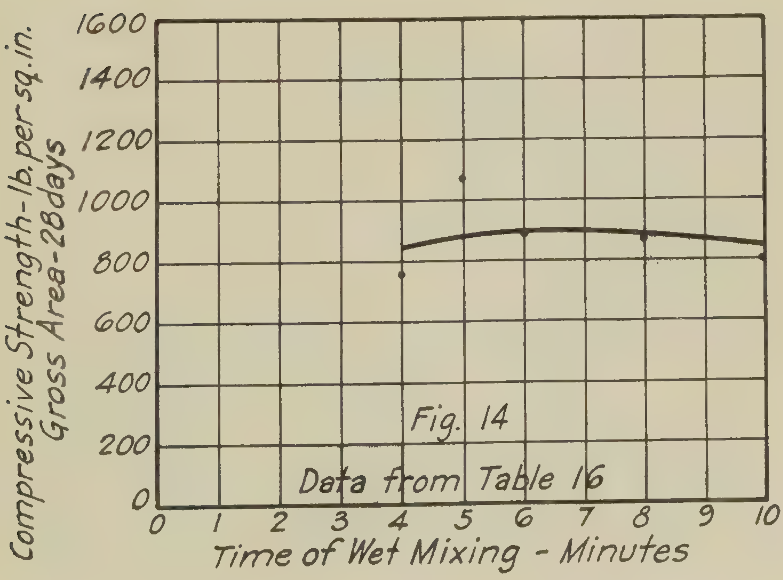
TIME OF MIXING TEST CONDUCTED AT OMAHA, NEBRASKA

Submitted by Committee P-6

This supplement presents the known facts covering the time of mixing test which was one of a series of experiments on methods of manufacturing concrete block conducted at the Omaha Concrete Stone Company during August and September, 1926. The units were tested at the Omaha Testing Laboratory.

The specimens were made under regular plant operating conditions, the only departure being that the materials were proportioned by the engineers in charge of the tests. The mixer was of the batch type with


Compressive Strength-lb.per sq.in.
Gross Area-28 days
1600
1400
1200
1000
800
600
400
200
0
0 1 2 3 4 5 6 7 8 9 10
Time of Wet Mixing - Minutes
Fig. 13
Data from Table 15


Compressive Strength-lb.per sq.in.
Gross Area-28 days
1600
1400
1200
1000
800
600
400
200
0
0 1 2 3 4 5 6 7 8 9 10
Time of Wet Mixing - Minutes
Fig. 14
Data from Table 16

revolving paddles. The block machine was of the stripper type, with four full-width tamper feet making standard 8 x 8 x 16-in. hollow block. An average temperature of 95 deg. F. was maintained in the curing rooms.

The following paragraphs discussing the results of this test are quoted from the original report written by the engineer in charge of the tests.

"In the group of tests to show the effect of time of mixing, a 1:7 mix by volume of dry and rodded aggregate was used with the grading of aggregate maintained constant (F. M. 3.8). All batches were mixed 2 min. dry and then wet to the total indicated (see Table 16). Total mixing times of 4, 5, 6, 8 and 10 min. were used. The results are plotted in Fig. 14. A total mixing time less than 4 min. was not used because this for the 1-sack mixers, operated by one man, used for each of four machines would be the shortest time possible with uniform operations throughout and would only produce excess of mixed concrete stored on floor back of machine allowing vaporation and possible retrogression.

"The result for the 5-min. mix is probably high. No material benefit is shown for mixing time above 5 min.; with a total mixing time of 4 min. a reduction in possible strength seems evident.

"Thorough mixing adds to the workability and strength of concrete in plastic concretes. Similar and beneficial results therefore, would seem probable in dry mixtures used in products manufacture."

TABLE 16—EFFECT OF TIME OF MIXING ON THE COMPRESSIVE STRENGTH OF CONCRETE BLOCK.

Tests of 8 x 8 x 16-in. concrete building block.
Aggregate—Platte River sand (0–No. 4), gravel (No. 14–¾ in.).
Fineness modulus of mixed aggregate = 3.9.
Mix 1:7 by volume dry and rodded aggregate.
Machine mixed concrete.
Time of mixing shown includes 2 min. of dry mixing.
Number of tamps each of four foot—14.
Curing: 48 hr. in steam; remainder in air.
Age at test, 28 days.
Water-cement ratio—0.98.
Blocks per sack of cement—22.
Each value of strength is the average of 3 tests, and of absorption, 2 tests.

Ref.	Time of Mixing, minutes	Compressive Strength, lb. per sq. in.		Absorption, Per Cent by Dry Weight	
		Gross Area	Net Area	24 Hr.	28 Hr.
10	4	757	1235	3.64	3.64
1	5	1068*	1743	2.80	3.62
11	6	885	1442	3.35	3.63
12	8	867	1413	3.37	3.66
13	10	795	1298	3.33	3.33

THE OPERATING EFFICIENCY OF CONCRETE PRODUCTS PLANTS

Submitted by Committee P-6

Committee P-6 originally intended to conduct a thorough investigation into the operating efficiency of concrete products plants to determine how many units can be produced per thousand dollars of invested capital and per man employed, but the investigation on wet and dry mixing periods occupied so much time that only a preliminary study was made.

In a certain mid-western city there are 26 concrete products plants with an invested capital in buildings, equipment and land of $500,000. These plants produced in 1928 the equivalent of 4,700,000 8 x 8 x 16-in. concrete block. Three of the 26 plants produced 49 per cent of the total output and four failed to turn a wheel. The production varied from nothing to a million units with half the plants producing 100,000 units or more. The average production was 181,500 units.

Production per $1,000 of invested capital ranged from 2,000 to 53,000 units with only eight plants producing 10,000 units or over. Ten of the plants produced less than 5,000 units per $1,000 of invested capital.

The fixed overhead was made up as follows: (1) depreciation and obsolescence of equipment, 20 per cent (2) annual interest on the investment, 6 per cent (3) taxes, ¼ per cent. Minimum power charge at $1 per h.p. was estimated as closely as possible. The fixed overhead costs varied from $6,000 per unit for one of the non-producing plants to only 3.6 mills for an exceptionally efficient plant. The highest charge for a producing plant was $.0934. Sixteen of the plants had a charge of more than 2 cents per unit. The plant with a fixed overhead of only 3.6 mills has a total investment of $15,000 and is equipped with two automatic block machines. This plant produced 798,700 units last year.

This information is presented to show the wide range in operating efficiencies of concrete products plants. Much research has been conducted on combinations of available aggregates to produce the strongest unit for a given mix, on relation of mixing time to unit strength, on the relation of tamping to unit strength and on the effect of various curing temperatures on the 28-day strength of concrete masonry units. Most of this information is now being used by leading manufactures and it appears that the most fertile field for producing information which will assist manufacturers remodeling old plants or erecting new ones lies in a study of the most efficient methods of using the invested capital and managing labor, in fact, plant management.

Since it is pertinent to the future welfare of the industry that manufacturing costs be reduced and since this survey indicates that the labor and fixed overhead costs offer larger opportunities for reduction than material costs the results of a thorough survey should be of help to every manufacturer.

Standard Building Units

Report of Committee P-1

Your committee on Standard Concrete Building Units cooperared with the American Society for Testing Materials and the Federal Specifications Board in an effort to obtain uniform requiemtents in specifications for concrete brick.

A meeting to discuss these requirements and specifications was held at Troy, New York, November 30, 1928, Professor T. R. Lawson representing the American Society for Testing Materials, S. H. Ingberg representing the United States Bureau of Standards and E. Grant Lantz representing the American Concrete Institute. A rough specification was prepared which has been discussed and subjected to various changes. Those attending that meeting believe that the present draft is worthy of inclusion in the Committee P-1 report so that the requirements can be opened to general discussion. A copy of these requirements will also be submitted to Committee C-3 of the American Society for Testing Materials and to the Federal Specifications Board.

The committee originally intended to continue the preliminary investigation of methods of tamping and feeding employed in the manufacture of tamped concrete masonry units reported by this Committee last year. But at a joint meeting of Committee P-1 and P-6 it was decided that this investigation was properly the work of Committee P-6 and further activity by this committee was discontinued.

On this report 15 members of the Committee voted affirmatively, 1 negatively, and 3 refrained from voting.

E. Grant Lantz, *Secretary.*

Requirements for Concrete Brick

Scope

1. These specifications cover concrete building brick intended for use in brick masonry.

Manufacture

2. Brick under this specification shall be made of portland cement mortar or concrete. They shall be sound, of compact structure, reasonably uniform in shape, and free from injurious materials in kinds and amounts that would impair durability or strength.

Physical Properties

3. (*a*) Concrete building brick shall be classified as Grades A, B, or C on the basis of the following requirements. The classification of any lot of brick shall be determined by the results of the tests for that requirement in which it is lowest. (*b*) The standard size of concrete building brick shall be $2\frac{1}{4} \times 3\frac{3}{4} \times 8$ in. with a permissible variation of plus or minus $\frac{1}{16}$ in. in depth, $\frac{1}{8}$ in. in width and $\frac{1}{4}$ in. in length from the standard size.

	Transverse Breaking Load in lb. 7-in. Span		Weight of Water Absorbed per Whole Brick	
	Mean of 5 Tests	Individual Minimum	Mean of 5 Tests	Individual Maximum
Grade A..........	1080 or more	725	6 oz. (170.1 g.)	8 oz. (226.8 g.)
Grade B..........	810 to 1080	540	8 oz. (226.8 g.)	10 oz. (283.5 g.)
Grade C..........	540 to 810	360	10 oz. (283.5 g.)	12 oz. (340.2 g.)

NOTE.—Brick in Grade C are not intended for use in the construction of exterior walls unless faced with suitable masonry veneer or portland cement stucco. Brick in Grade A are intended for use as face brick.

Developed by joint meeting of Committee C-3 of the A.S.T.M., S. H. Ingberg of the Federal Specifications Board and Committee P-1 of the A.C.I.

FLEXURE TEST

Test Specimen

4. The test specimen shall be a whole brick.

Number of Tests

5. Flexure tests shall be made on at least five whole brick.

Procedure

6. Preparatory to the flexure test the samples shall be dried in a drier or oven at a temperature of from 212 to 220 deg. F. until the loss in weight does not exceed 2 gr. after 2 hr. of drying.

7. (*a*) When cool, 5 of the 10 brick in the sample selected shall be tested, laid flatwise on a span of 7 inches and with the load applied at the mid-point of the span with a standard testing machine or calibrated portable or semi-portable testing equipment. A steel bearing plate about $\frac{1}{4}$ in. thick by $1\frac{1}{2}$ in. wide shall be placed between the upper knife-edge and the brick. The knife-edges in contact with the brick shall be mounted so they will adjust themselves to the irregularities in the shape of the brick, and one or both of the lower bearings shall be free to follow any movement of the brick during the test.

NOTE.—Portable apparatus properly calibrated and accurate and sensitive within 30 lb. up to 2,000 lb. applied load, and within 60 lb. for higher loads may be used, although laboratory equipment should be used where available.

(*b*) The speed of travel of the head of the testing machine, running idle, shall not be more than 0.05 in. per minute.

ABSORPTION TEST

Test Specimen

8. The test specimen shall be a whole brick.

Number of Tests

9. Absorption tests shall be made on at least five whole brick.

Procedure

10. Preparatory to the absorption test the samples shall be dried in a drier or oven at a temperature of from 212 to 220 deg. F. until the loss in weight does not exceed 2 gr. after 2 hr. drying.

11. The balance used shall be sensitive to within 0.2 per cent of the lightest specimen tested.

12. After obtaining the dry weight of the samples, they shall be immersed in soft, distilled or rain water at room temperature (70 deg. F.) for 5 hours.

13. After immersion, the sample shall be removed from the water and allowed to drain for not more than 1 minute. The superficial water shall be removed with a damp cloth, after which they shall be weighed immediately.

14. The test results shall be expressed as ounces or grams of water absorbed, carried to the nearest first decimal. The results shall be reported separately for each brick, with the average for the five brick.

Sampling

15. For the purpose of tests, brick representative of the commercial product shall be selected by a competent person appointed by the purchaser, the place or places of selection to be designated when the purchase order is placed. The manufacturer or seller shall furnish specimens for test without charge. All brick shall be carefully examined and their condition noted before testing.

For the purpose of tests, not less than 10 brick shall be required. In general, 2 samples of 10 brick each shall be tested for every 100,000 brick contained in the lot under consideration; but where the total quantity exceeds 500,000, the number of samples tested per 100,000 brick may be fewer, provided they shall be distributed as uniformly as practicable over the entire lot. Additional representative samples may be taken at any time or place at the discretion of the purchaser, providing the brick are 28 days old or of the same age as those being delivered on the job.

Inspection and Rejection

16. Concrete building brick shall pass a visual inspection for freedom from cracks and irregularity. No shipment shall contain more than 5 per cent of broken brick.

Discussion—Standard Building Units

E. G. Lantz—I have two specifications up for vote—a specification for Concrete Sewer Manhole and Catch Basin Block and a specification for Concrete Block and Tile. I move that the specification (Pl-C-28 T) for Concrete Sewer Manhole and Catch Basin Block be moved up from a tentative to a standard specification. (Note—the motion carried and it was voted to refer the tentative specification to letter ballot for advancement to standard.) **Mr. Lantz**

E. G. Lantz—I move that the specification for Concrete Block and Tile be moved from tentative to full standard. (Note—The motion carried and it was voted to refer the tentative specification to letter ballot for advancement to standard.) **Mr. Lantz.**

Joint Concrete Culvert Pipe Committee*

Second Report

The Joint Concrete Culvert Pipe Committee was organized in 1919 to prepare standard specifications for reinforced concrete pipe to be used for railway and highway culverts. The committee has held one or more meetings each year since its organization and in February, 1926, it presented to the constituent organizations a report embodying recommended tentative standard specifications for reinforced concrete culvert pipe. That report was very widely distributed; more than 5,000 copies being furnished to public officials and manufacturers and users of pipe in the United States and Canada.

At the outset the committee found that there was little known with reference to the actual loads to which culverts are subjected. An investigation of this subject had been inaugurated at Iowa State College under the direction of Dean Anson Marston. The committee encouraged the vigorous prosecution of that work so that the results could be used in connection with the development of the specifications for culvert pipe. One meeting of the committee was held at Ames at which time the research under way was reviewed and the whole problem of loads was given extended consideration. At later times, two progress reports on the Ames investigation were submitted to the committee. The data included were of great value in connection with the determination of the loads for which culvert pipe should be designed.

A considerable amount of data from tests of pipe and from experience in the use of pipe were furnished by members of the committee and by manufacturers of pipe and engineers who had installed pipe or observed its behavior in embankments. All of this material was considered by the committee when the first tentative specifications were being prepared.

After the 1926 specifications were distributed the committee accumulated evidence indicating that pipe could be produced to meet the strength requirements with less circumferential reinforcement than was prescribed by the specifications. Accordingly the committee arranged for a series of tests to be made under the direction of W. J. Schlick, Drainage Engi-

*Constituent Organizations:
American Concrete Institute—Committee J-2.
American Society for Testing Materials.
Bureau of Public Roads, U. S. Department of Agriculture.
American Society of Civil Engineers.
American Association of State Highway Officials.
American Railway Engineering Association.
American Concrete Pipe Association.

neer, Iowa Engineering Experiment Station, for the purpose of securing information with reference to this subject. Pipe with various amounts of circumferential reinforcement and made of concrete of various strengths, were made and tested under the direction of Mr. Schlick in 8 pipe plants in the United States. There were 167 standard sections ranging from 12 to 60 inches in diameter. These tests showed that pipe of the specified strengths could be made with less circumferential reinforcement than was required by the 1926 specifications.

Meanwhile there had developed a demand for the inclusion of an absorption specification and in 1927 the Committee arranged for Mr. Schlick to test for absorption specimens of concrete pipe furnished by manufacturers in various parts of the United States.

Copies of the reports on the two series of tests may be obtained from the Secretary of the American Concrete Pipe Association, 33 West Grand Avenue, Chicago, Illinois.

The information secured from the tests and the additional facts that have been developed with reference to loads subsequent to the previous reports were the basis of the accompanying revision of the 1926 specifications. The principal revisions are in the amount of circumferential reinforcement, the load requirement for Extra Strength Pipe and the inclusion of an absorption test.

The moment formula given in the section on design is essentially the one developed years ago for an elastic ring under opposed external forces. A discussion of this theory was presented by Prof. A. N. Talbot of the University of Illinois in Bul. 22 of the Illinois Engineering Experiment Station entitled, "Tests of Cast Iron and Reinforced Concrete Culvert Pipe," which is now out of print.

Early in its deliberations the committee decided that, while culvert pipe may be used under many conditions of loading, it is feasible to meet all ordinary commercial needs with two stock, or standard, classes of pipe. The formulas for design provide a means of designing pipe to meet any special condition of loading for which standard pipe are not adapted.

The assumptions as to load for the two classes of pipe have been correlated with the strength test requirements and were adopted after considering the results of strength tests of pipe and measurements of actual loads on pipe.

The work of the committee was greatly facilitated by the cordial cooperation of manufacturers of culvert pipe. The industry assisted the committee in many ways and several manufacturers furnished pipe for testing and facilities at their plants for making the required tests.

It is the recommendation of this committee that the specifications submitted herewith be adopted by each constituent organization as tentative standard specifications for reinforced concrete culvert pipe, superseding those reported in February, 1926.

TENTATIVE STANDARD SPECIFICATIONS FOR REINFORCED CONCRETE CULVERT PIPE*

I. GENERAL

Scope—

1. These specifications apply to reinforced concrete pipe intended to be used for the construction of culverts.

Classes—

2. Pipe, under these specifications, shall be of two classes known respectively as *Standard Reinforced Concrete Culvert Pipe* and *Extra Strength Reinforced Concrete Culvert Pipe.*

Basis of Acceptance—

3. The acceptability of pipe shall be determined by the results of the strength and absorption tests hereinafter specified, if and when required, and by inspection to determine whether the pipe comply with the specifications as to design and freedom from defects.

II. MATERIALS

Reinforced Concrete—

4. The reinforced concrete shall consist of portland cement, mineral aggregate and water in which steel has been embedded in such a manner that the steel and the concrete act together in resisting forces.

Cement—

5. Portland cement shall meet the requirements of the current Standard Specifications and Tests for Portland Cement of the American Society for Testing Materials.

Steel—

6. Reinforcement may consist of wire which meets the requirements of the current Specifications for Cold-Drawn Steel Wire for Concrete Reinforcement of the American Society for Testing Materials, or of bars which meet the requirements of the current Standard Specifications for Billet-Steel Concrete Reinforcement Bars of the American Society for Testing Materials.

Fine Aggregate—

7. (*a*) Fine aggregate shall consiste of sand, stone screenings, or other inert materials with similar characteristics, or a combination thereof, having clean, hard, strong, durable, uncoated grains and free from injurious amounts of dust, lumps, soft or flaky particles, shale, alkali, organic matter, loam or other deleterious substances. Fine aggregate shall be well graded and shall pass a ¼-in. screen.

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* Approved by the convention as a Tentative Standard Specification (J-2A-29T) of the American Concrete Institute as presented by Committee J-2.

Coarse Aggregate—

(*b*) Coarse aggregate shall consist of crushed stone, gravel, slag, or other approved inert materials with similar characteristics, or combinations thereof, having clean, hard, strong, durable, uncoated particles, free from injurious amounts of soft, friable, thin, elongated or laminated pieces, alkali, organic or other deleterious matter.

Mixture—

8. The aggregates shall be so graded and proportioned and thoroughly mixed with such a proportion of cement and water as will produce a homogeneous concrete mixture of such quality that the concrete will meet the test and design requirements herein specified.

III. DESIGN

Methods—

9. The pipe shall be designed in accordance with the following assumptions:

(*a*) That the design load is equivalent to a vertical load uniformly distributed over the internal horizontal projection of the pipe, that the pipe is likewise uniformly supported and that no allowance is made for side pressure.

(*b*) The uniform load for the Standard Reinforced Concrete Culvert Pipe shall be 2000 lb. and for the Extra Strength Reinforced Concrete Culvert Pipe 4000 lb. per sq. ft. respectively.

(*c*) The working stress per sq. in. in compression for the concrete shall not exceed three-eighths of the strength of concrete upon which the design is based.

(*d*) The ratio (n) of the modulus of elasticity of steel to that of concrete shall be 12 for concrete having an ultimate compressive strength at 28 days of 2,750 lb. per sq. in. and 9 for concrete having an ultimate compressive strength of 4,000 lb. per sq. in. or greater. Intermediate values of n shall be proportional to the strength of concrete assumed in the design.

(*e*) The working stress for cold-drawn steel wire shall not exceed 27,500 lb. per sq. in. For billet-steel, intermediate and hard grades, the working stress shall not exceed 20,000 lb. per sq. in.; and for billet-steel, structural grade, the working stress shall not exceed 18,000 lb. per sq. in.

(*f*) The distance from the center of the reinforcement to the nearest or tension surface of the concrete shall not be less than ¾ inch for pipe 12 in. or less in diameter, or less than one inch for pipe more than 12 in. in diameter.

(*g*) The distance from the center of the tension reinforcement to the compression surface of the concrete and the area of the reinforcement shall not be less than that required by the formula—

$$\frac{wd}{16} \times \frac{d + t}{12} = jAtf_s$$

in which w = uniform vertical load in pounds per square foot top and bottom of pipe

d = internal diameter of pipe in inches

t = distance from the center of the tension reinforcement to the compression surface of the concrete in inches

A = sectional area of tension reinforcement in square inches per lineal foot of the pipe

f_s = tensile stress in the reinforcement in pounds per square inch

j = ratio of the lever arm of the reinforcement to t as determined by the usual formulas

Minimum Designs—

10. The shell thickness and the amount of circumferential reinforcement shall not be less than that given in the design tables for the classes and sizes of pipe and the strength of concrete therein specified.

Alternative Designs—

11. Manufacturers may submit to the consumer or purchaser, for approval, designs based on strengths of concrete other than those given in the design tables. Such alternate designs shall comply with the design requirements given in Section III of these specifications. In no alternative design, however, shall the shell thicknesses be less than those given in Table II, nor shall the strength of concrete be less than that given in Table I.

Standard Sizes—

12. Pipe of the internal diameters listed in the design tables shall be considered standard sizes for culvert construction. In elliptical pipe, the inside diameter at the minor axis shall be equal to the diameter of the corresponding size of circular pipe.

Joints—

13. The ends of the pipe shall be of such design that the pipe when laid shall make a continuous conduit with a smooth and uniform interior surface.

Placing Reinforcement—

14. When a single line of circular reinforcement is used n circular pipe, it shall be placed at the center of the pipe shell. When two lines of reinforcement are used in circular pipe, one shall be placed near the inner and one near the outer surface of the pipe. The single line of elliptical reinforcement used in circular pipe, or the single line of circular reinforcement in elliptical pipe shall be placed near the inner surface at the "top" and "bottom" of the pipe and near the outer surface at the sides (see Paragraph 20 (*d*)).

TABLE I—DESIGNS OF STANDARD REINFORCED CONCRETE CULVERT PIPE

Uniform load of 2000 lb. per sq. ft. Ultimate compressive strength of concrete, 2750 lb. per sq. in.
(f_c = 1030 lb.)

Internal Diameter of Pipe in inches d	Minimum Thickness of Shell in inches	Minimum Distance Center of Reinforcement to Compressive Surface in inches t		Minimum Area of Circular Reinforcement, sq. in. per lin. ft. of Pipe "A"			
				Cold Drawn Steel Wire, f_s = 27,500 lb. per sq. in.		Billet Steel Hard and Intermediate Grades, f_s = 20,000 lb. per sq. in.	
		Circular Reinforcement in Circular Pipe	Elliptical Reinforcement in Circular Pipe and Circular Reinforcement in Elliptical Pipe	Circular Reinforcement in Circular Pipe	Elliptical Reinforcement in Circular Pipe and Circular Reinforcement in Elliptical Pipe	Circular Reinforcement in Circular Pipe	Elliptical Reinforcement in Circular Pipe and Circular Reinforcement in Elliptical Pipe
12	2	1	...	1 Line .07	....	1 Line .09	....
15	2¼	1⅛	1¼	1 " .09	1 Line .08	1 " .13	1 Line .12
18	2½	1¼	1½	1 " .12	1 " .10	1 " .17	1 " .14
24	3	1½	2	1 " .17	1 " .13	1 " .25	1 " .19
30	3½	1¾	2½	1 " .23	1 " .17	1 " .32	1 " .23
30	3½	2½	2½	2 " ea. .17	1 " .17	2 " ea. .23	1 " .23
36	4	3	3	2 " " .20	1 " .20	2 " " .28	1 " .28
42	4½	3½	3½	2 " " .23	1 " .23	2 " " .32	1 " .32
48	5	4	4	2 " " .26	1 " .26	2 " " .37	1 " .37
54	5½	4½	4½	2 " " .30	1 " .30	2 " " .42	1 " .42
60	6	5	5	2 " " .33	1 " .33	2 " " .46	1 " .46
72	7	6	6	2 " " .40	1 " .40	2 " " .56	1 " .56
84	8	7	7	2 " " .46	1 " .46	2 " " .65	1 " .65

TABLE II—DESIGNS OF STANDARD REINFORCED CONCRETE CULVERT PIPE

Uniform load of 2000 lb. per sq. ft. Ultimate compressive strength of concrete, 4000 lb. per sq. in.
(f_c = 1500 lb.)

Internal Diameter of Pipe in inches d	Minimum Thickness of Shell in inches	Minimum Distance Center of Reinforcement to Compressive Surface in inches t		Minimum Area of Circular Reinforcement, sq. in. per lin. ft. of Pipe "A"			
				Cold Drawn Steel Wire, f_s = 27,500 lb. per sq. in.		Billet Steel Hard and Intermediate Grades, f_s = 20,000 lb. per sq. in.	
		Circular Reinforcement in Circular Pipe	Elliptical Reinforcement in Circular Pipe and Circular Reinforcement in Elliptical Pipe	Circular Reinforcement in Circular Pipe	Elliptical Reinforcement in Circular Pipe and Circular Reinforcement in Elliptical Pipe	Circular Reinforcement in Circular Pipe	Elliptical Reinforcement in Circular Pipe and Circular Reinforcement in Elliptical Pipe
12	1¾	⅞	...	1 Line .08	....	1 Line .11	....
15	2	1	...	1 " .11	....	1 " .15	....
18	2¼	1⅛	1¼	1 " .14	1 Line .12	1 " .19	1 Line .17
24	2⅝	1¼	1⅝	1 " .21	1 " .17	1 " .30	1 " .23
30	3	1½	2	1 " .29	1 " .21	1 " .38	1 " .29
30	3	2	2	2 " ea. .21	1 " .21	2 " ea. .29	1 " .29
36	3⅜	2⅜	2⅜	2 " " .26	1 " .26	2 " " .36	1 " .36
42	3¾	2¾	2¾	2 " " .30	1 " .30	2 " " .41	1 " .41
48	4¼	3¼	3¼	2 " " .34	1 " .34	2 " " .46	1 " .46
54	4⅝	3⅝	3⅝	2 " " .38	1 " .38	2 " " .52	1 " .52
60	5	4	4	2 " " .42	1 " .42	2 " " .59	1 " .59
72	5¾	4¾	4¾	2 " " .51	1 " .51	2 " " .71	1 " .71
84	6⅝	5⅝	5⅝	2 " " .60	1 " .60	2 " " .82	1 " .82

TABLE III—DESIGNS OF EXTRA STRENGTH REINFORCED CONCRETE CULVERT PIPE

Uniform load of 4000 lb. per sq. ft. Ultimate compressive strength of concrete, 2750 lb. per sq. in.
(f_c = 1030 lb.)

Internal Diameter of Pipe in inches d	Minimum Thickness of Shell in inches	Minimum Distance Center of Reinforcement to Compressive Surface in inches t		Minimum Area of Circular Reinforcement, sq. in. per lin. ft. of Pipe "A"			
				Cold Drawn Steel Wire, f_s = 27,500 lb. per sq. in.		Billet Steel Hard and Intermediate Grades, f_s = 20,000 lb. per sq. in.	
		Circular Reinforcement in Circular Pipe	Elliptical Reinforcement in Circular Pipe and Circular Reinforcement in Elliptical Pipe	Circular Reinforcement in Circular Pipe	Elliptical Reinforcement in Circular Pipe and Circular Reinforcement in Elliptical Pipe	Circular Reinforcement in Circular Pipe	Elliptical Reinforcement in Circular Pipe and Circular Reinforcement in Elliptical Pipe
12	2½	1¼	1½	1 Line .11	1 Line .09	1 Line .15	1 Line .13
15	2¾	1⅜	1¾	1 " .15	1 " .12	1 " .21	1 " .17
18	3⅛	1⅝	2⅛	1 " .18	1 " .14	1 " .26	1 " .20
24	3⅞	2	2⅞	1 " .26	1 " .19	1 " .37	1 " .27
30	4⅝	2¼	3⅝	1 " .36	1 " .23	1 " .51	1 " .33
30	4⅝	3⅝	3⅝	2 " ea. .23	1 " .23	2 " ea. .33	1 " .33
36	5⅜	4⅜	4⅜	2 " " .28	1 " .28	2 " " .40	1 " .40
42	6	5	5	2 " " .33	1 " .33	2 " " .47	1 " .47
48	6¾	5¾	5¾	2 " " .38	1 " .38	2 " " .53	1 " .52
54	7½	6½	6½	2 " " .42	1 " .42	2 " " .60	1 " .60
60	8¼	7¼	7¼	2 " " .47	1 " .47	2 " " .66	1 " .66
72	9⅝	8⅝	8⅝	2 " " .57	1 " .57	2 " " .80	1 " .80
84	11	10	10	2 " " .67	1 " .67	2 " " .94	1 " .94

TABLE IV—DESIGNS OF EXTRA STRENGTH REINFORCED CONCRETE CULVERT PIPE

Uniform load of 4000 lb. per sq. ft. Ultimate compressive strength of concrete, 4750 lb. per sq. in.
(f_c = 1780 lb.)

Internal Diameter of Pipe in inches d	Minimum Thickness of Shell in inches	Minimum Distance Center of Reinforcement to Compressive Surface in inches t		Minimum Area of Circular Reinforcement, sq. in. per lin. ft. of Pipe "A"			
				Cold Drawn Steel Wire, f_s = 27,500 lb. per sq. in.		Billet Steel Hard and Intermediate Grades, f_s = 20,000 lb. per sq. in.	
		Circular Reinforcement in Circular Pipe	Elliptical Reinforcement in Circular Pipe and Circular Reinforcement in Elliptical Pipe	Circular Reinforcement in Circular Pipe	Elliptical Reinforcement in Circular Pipe and Circular Reinforcement in Elliptical Pipe	Circular Reinforcement in Circular Pipe	Elliptical Reinforcement in Circular Pipe and Circular Reinforcement in Elliptical Pipe
12	2	1	...	1 Line .14	....	1 Line .19	....
15	2¼	1⅛	1¼	1 " .19	1 Line .17	1 " .26	1 Line .24
18	2½	1¼	1½	1 " .24	1 " .21	1 " .34	1 " .29
24	3	1½	2	1 " .35	1 " .27	1 " .50	1 " .39
30	3½	1¾	2½	1 " .47	1 " .34	1 " .66	1 " .48
30	3½	2½	2½	2 " ea. .34	1 " .34	2 " ea. .48	1 " .48
36	4	3	3	2 " " .41	1 " .41	2 " " .57	1 " .57
42	4½	3½	3½	2 " " .48	1 " .48	2 " " .67	1 " .67
48	5	4	4	2 " " .55	1 " .55	2 " " .76	1 " .76
54	5½	4½	4½	2 " " .62	1 " .62	2 " " .86	1 " .86
60	6	5	5	2 " " .68	1 " .68	2 " " .95	1 " .95
72	7	6	6	2 " " .82	1 " .82	2 " " 1.14	1 " 1.14
84	8	7	7	2 " " .96	1 " .96	2 " " 1.33	1 " .133

Longitudinals—

15. Each line of circumferential reinforcement shall be assembled into a cage and have sufficient longitudinal bars or members, extending through the barrel of the pipe, to afford rigidity and maintain the reinforcement in exact shape and correct position within the form.

Laps and Welds—

16. The reinforcement shall be lapped not less than 30 diameters, or if welded, the joints shall develop the full strength of the reinforcement. The spacing center to center of adjacent rings of circumferential reinforcement in a cage shall not exceed 4 in. up to and including pipe 48 in. in diameter, nor exceed the shell thickness for larger pipe and shall in no case exceed 6 inches.

Bell Reinforcement—

17. The bell shall have a circumferential reinforcement equal in unit area to that of a single line within the barrel of the pipe.

IV. WORKMANSHIP AND FINISH

Finish—

18. Pipe shall be substantially free from fractures, large or deep cracks and surface roughness. The planes of the ends of the pipe shall be perpendicular to their longitudinal axes.

Variations in Dimensions—

19. (*a*) Variations of the internal diameter shall not exceed 1½ per cent nor shall the shell thickness be less than that intended in the design by more than 5 per cent at any point.

(*b*) Variation in the position of the reinforcement cages shall not exceed ¼ in. from the position provided in the design, nor shall the cover on the reinforcement be less than ¾ in. at any point.

V. MARKING

Markings—

20. The following shall be clearly stenciled on the pipe:

(*a*) The pipe class $\begin{cases}\text{by an “S” for Standard Pipe}\\ \text{and an “X” for Extra Strength Pipe}\end{cases}$

(*b*) The date of manufacture

(*c*) The name or trade-mark of manufacturer

(*d*) Elliptical pipe with circular reinforcing and circular pipe with elliptical reinforcing shall have the words "Top or Bottom" clearly stenciled on the inside of the pipe at the correct place to indicate the proper position when laid.

VI. PHYSICAL TESTS

Strength Tests—

21. Pipe may be tested for strength by either the three-edge or sand bearing method:

Three-Edge Bearing Test—

(*a*) When the three-edge bearing is used, the lower bearing for the pipe shall consist of two wooden strips with vertical sides having their interior top corners rounded to a radius of approximately ½ in. The strips shall be straight and shall be securely fastened to a rigid block with the interior vertical sides spaced a distance apart not less than ½ in. nor more than 1 in. for each foot of diameter pipe. The upper bearing shall be a rigid wooden block, straight and true from end to end. The upper and lower bearings shall extend the full length of pipe exclusive of bell. The pipe shall be placed symmetrically between the two

TABLE V—MINIMUM STRENGTH OF REINFORCED CONCRETE CULVERT PIPE POUNDS PER FT. OF LAYING LENGTH

Size of Pipe, in.	Standard Reinforced Concrete Culvert Pipe				Extra Strength Reinforced Concrete Culvert Pipe			
	3 Edge Bearing		Sand Bearing		3 Edge Bearing		Sand Bearing	
	Cracking Load*	Ultimate Load	Cracking Load*	Ultimate Load	Cracking Load*	Ultimate Load	Cracking Load*	Ultimate Load
12	1,600	2,000	2,400	3,000	3,200	4,000	4,800	6,000
15	1,800	2,500	2,700	3,750	3,600	5,000	5,400	7,500
18	2,000	3,000	3,000	4,500	4,000	6,000	6,000	9,000
24	2,200	4,000	3,300	6,000	4,400	8,000	6,600	12,000
30	2,500	5,000	3,750	7,500	5,000	10,000	7,500	15,000
36	3,000	6,000	4,500	9,000	6,000	12,000	9,000	18,000
42	3,500	7,000	5,250	10,500	7,000	14,000	10,500	21,500
48	4,000	8,000	6,000	12,000	8,000	16,000	12,000	24,000
54	4,500	9,000	6,750	13,500	9,000	18,000	13,500	27,000
60	5,000	10,000	7,500	15,000	10,000	20,000	15,000	30,000
72	6,000	12,000	9,000	18,000	12,000	24,000	18,000	36,000
84	7,000	14,000	10,500	21,000	14,000	28,000	21,000	42,000

* At the cracking load there shall be, in the barrel of the pipe, no crack having a surface width of .01 inch or more for a length of one foot or more.

bearings as illustrated in Figs. 3 and 4. In testing pipe which is "out of line" the lines of the bearings chosen shall be from those which appear to give the most favorable conditions for fair test.

Sand Bearing Test—

(*b*) When sand bearings are used (see Figs. 2 and 5), the ends of each specimen of pipe shall be accurately marked prior to the test in quarters of the circumference. Specimens shall be carefully bedded, above and below, in sand, for one-fourth the circumference of the pipe measured on the middle line of the barrel. The depth of bedding above and below the pipe at the thinnest points shall be one-half the radius of the middle line of the barrel.

The sand used shall be clean and moist, and shall be such as will pass a 4760-micron sieve (U. S. Standard No. 4). The sand in the lower bearing shall be loose when the pipe is placed.

The top bearing frame shall not be allowed to come in contact with the pipe nor with the top bearing plate. The upper surface of the sand in the top bearing shall be stuck level with a straight edge, and shall be covered with a rigid top bearing plate, with lower surface a true plane, made of heavy timbers or other rigid material, capable of distributing the test load uniformly without appreciable bending. The test load shall be applied at the exact center of this top bearing plate, or in such manner as to produce uniform deflection throughout the full length of the pipe. For this purpose a spherical bearing is preferred, but two rollers at right angles may be used. The test may be made without the use of a testing machine, by piling weights directly on a platform resting on the top bearing plate, provided, however, that the weights shall be piled symmetrically about a vertical line through the center of the pipe, and that the platform shall not be allowed to touch the top bearing frame.

The frames of the top and bottom bearings shall be made of timbers so heavy as to avoid appreciable bending by the side pressure of sand.

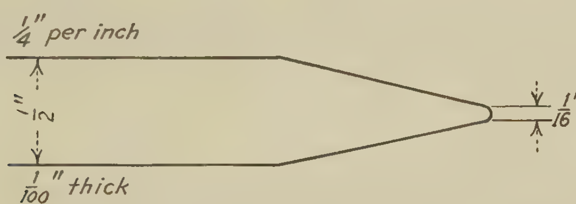

FIG. 1—GAGE FOR MEASURING CRACKS IN PIPE

The interior surfaces of the frames shall be dressed. No frame shall come in contact with the pipe during the test. A strip of cloth may, if desired, be attached to the inside of the upper frame on each side, along the lower edge, to prevent the escape of sand between the frame and the pipe.

Application of Load (Testing Machine)—

(*c*) It is desirable that a machine shall be used which gives a uniform deflection throughout the full length of the pipe. Any mechanical or hand power device may be used in which the head that applies the load moves at a speed of not more than 0.05 inches per minute while making the test. The testing machine shall be substantial and rigid throughout, so that the distribution of the load will not be affected appreciably by the deformation or yielding of any part. The load shall be applied continuously until the ultimate strength of the pipe is reached.

Strength Requirements—

22. The ultimate load, as determined by one of the methods described in Paragraph 21, shall not be less than the ultimate load specified in Table V for the size and class of pipe that is being tested. When the test load reaches the cracking load specified in Table V for the size and

class of pipe that is being tested, there shall be in the barrel of the pipe no crack having a surface width of 1/100 in.* or more, for a length of

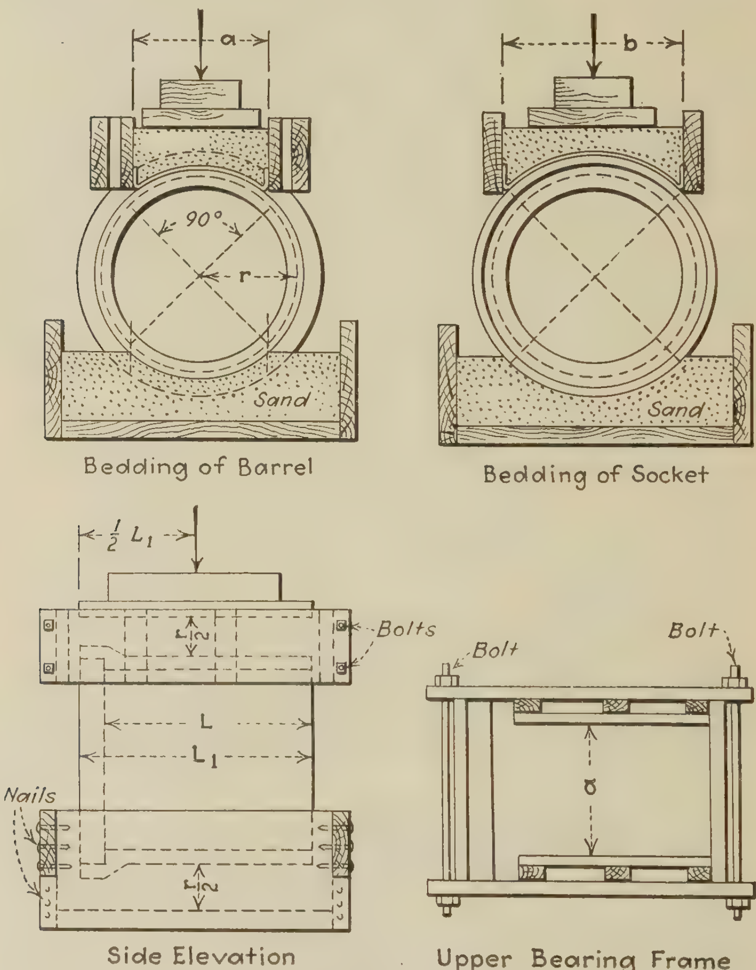

FIG. 2—SAND BEARING TESTS

one foot or more. The ultimate load is reached when the pipe will sustain no greater load.

* It is recommended that the width of the crack be measured by means of a gage made from a leaf 1/100-inch thick from a set of standard machinists' gages, ground to a point 1/16-inch wide, with corners rounded, and with a taper of ¼-inch per inch, as illustrated by Fig. 1. The crack shall be considered to be 1/100-inch wide when the point of the gage will just enter it at close intervals.

Elliptical Pipe—

23. Elliptical pipe shall meet the test requirements for circular pipe having the same horizontal internal diameter.

Preliminary Tests and Tests for Extended Deliveries—

24. Preliminary to placing an order, a consumer of pipe whose needs require shipments at intervals over extended periods of time shall be entitled to test not more than ten pieces of pipe covering the size in which he is interested. The test specimens shall be selected in approximately equal numbers from the larger and smaller sizes of pipe. The acceptability of the larger sizes of pipe shall not be based on the results of tests in smaller sizes. After these preliminary tests, a consumer shall be entitled to additional tests in such numbers and at such times as he may deem necessary, provided that the total number of pipe tested shall not exceed two per cent (2 per cent) of the total deliveries.

Tests for Occasional Orders—

25. A purchaser who places occasional orders shall be entitled to test a number of pipe equal to two per cent (2 per cent) of an order but not to exceed five pieces of any one size; otherwise the number of pipe desired for testing shall be included in the order.

Selecting Test Specimens—

26. All pipe for testing purposes shall be selected at random by the consumer or purchaser from the stock of the manufacturer and shall be pipe which would not otherwise be rejected under these specifications. The pipe shall be free from visible moisture when tested.

They shall not have been exposed to a temperature below 40 deg. (F.) for the 24 hr. immediately preceding the test.

Cylinder Tests and Reinforcement Examinations—

27. By agreement between the consumer and the manufacturer the continued acceptability of the pipe, after the preliminary pipe tests have been made, may be determined by tests of the quality of the concrete as placed in the pipe and examination of the quality, amount and the accuracy of placement of the reinforcement. The quality of the concrete shall be determined on 6 x 12-in. test cylinders taken from the concrete used in making the pipe and manufactured and cured under identical conditions with the pipe. When tested in accordance with the current standard methods prescribed by the American Society for Testing Materials, these cylinders shall have a strength not less than that assumed in the design of the pipe.

Retests—

28. Pipe shall be acceptable under the strength tests when all test specimens meet the test requirements. Should less than three of the ten preliminary test specimens or any one of the additional test speci-

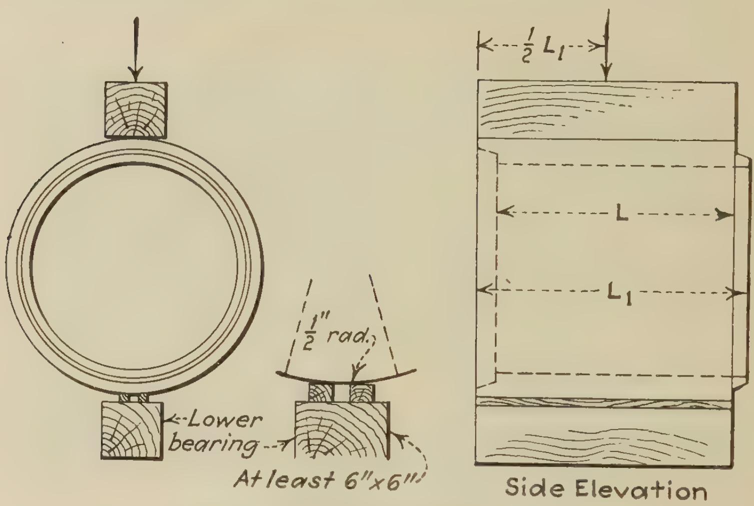

FIG. 3—THREE-EDGE BEARING TEST

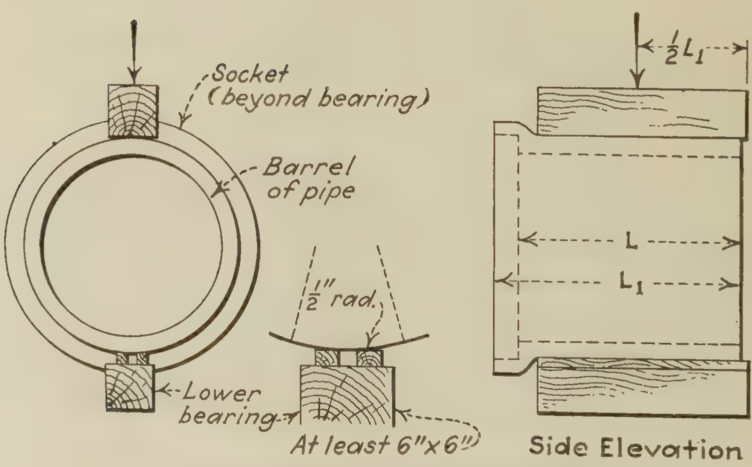

FIG. 4—THREE-EDGE BEARING TEST

mens provided for in Paragraph 24, or any one specimen provided for in Paragraph 25 fail to meet the test requirements, then the manufacturer will be allowed a retest on two like specimens for each specimen that failed, and the pipe shall be acceptable only when all of these retest specimens meet the test requirements. No further retests shall be permitted.

Test Specimens—

29. Absorption tests shall be made by the following method:

(*a*) The number of absorption specimens shall be equal to the number of pipe provided for testing. The specimens shall be obtained from pipe that are acceptable as to strength and shall be taken from pipe used in making the strength test when that test is made. The specimens shall be marked with the number or identifying mark of the pipe from which they were taken. Each specimen shall have an area of 16 to 24 sq. in. and a thickness equal to the full depth of the pipe shell, and shall be free from visible cracks.

Drying Specimens—

(*b*) Specimens shall be dried at a temperature of approximately 110 deg. C. (230 deg. F.) until no loss of weight is shown by successive weighings at intervals of not less than four hours.

Immersion and Reweighing—

(*c*) The dried specimens shall be placed in a suitable receptacle, covered with distilled water or rain water, raised to the boiling point and boiled for five hours, and then cooled in water to a final temperature of from 15 to 20 deg. C. (59 to 68 deg. F.). When cool, the specimens shall be removed from the water, allowed to drain for not more than one minute, the superficial water removed by a towel or blotting paper, and the specimens immediately weighed.

Weighing Devices—

(*d*) The balance used shall be sensitive to 0.5 g., when loaded with 1 kg. and weighings shall be read at least to the nearest gram. Where other than metric weights are used, the same degree of accuracy must be obtained.

Calculation and Reporting of Results—

(*e*) The increase in weight of the boiled specimen over its dry weight shall be considered the absorption of the specimen and shall be calculated as a percentage of the dry weight. The results shall be reported separately for each specimen.

Test Requirements and Acceptability Under Absorption Tests—

30. The absorption shall not exceed 8 per cent for test specimens taken from pipe designed to be made of concrete having a compressive

strength of 3000 or more lb. per sq. in., or 9 per cent for test specimens taken from pipe designed to be made of concrete having a compressive strength of less than 3000 lb. per sq. in. Pipe shall be considered to meet these specifications for absorption when not less than 80 per cent of the number of specimens tested, including any retested, meet the test requirements. When the initial absorption specimen from a pipe fails to meet these specifications, the absorption test shall be made on another specimen from the same pipe and the results of the retest shall be substituted for the original test results.

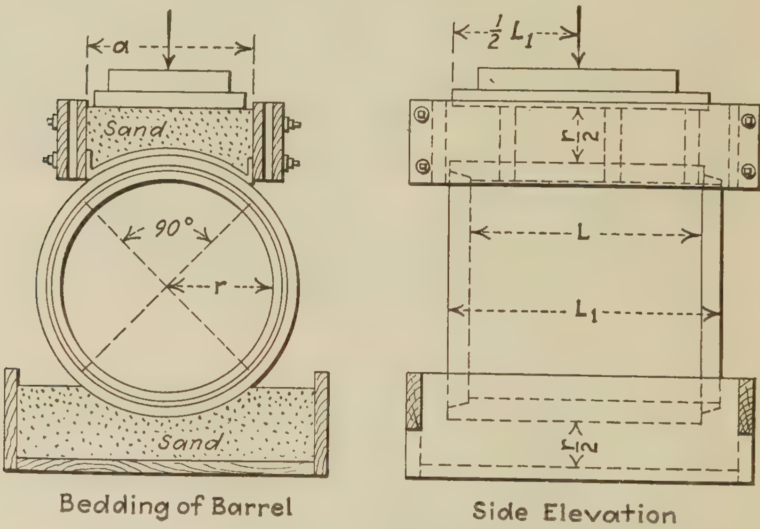

FIG. 5—SAND BEARING TEST

Minimum Age for Shipment—

31. Pipe will be considered ready for shipment when they meet the test requirements, or when tests of 6 x 12-in. cylinders (Section 27) show that the concrete has attained the strength assumed in the design of the pipe.

Test Equipment—

32. Every manufacturer furnishing pipe under these specifications shall furnish all facilities necessary to carry out the tests herein provided.

VII. INSPECTION

33. All materials, processes of manufacture and finished pipe shall be subject to inspection and approval by an inspector employed by the consumer or purchaser. The manufacturer when so directed by the

inspector shall have holes cut in such sections of the finished pipe (not exceeding one hole in every 50 sections delivered) as desired so that a proper inspection may be made of the quantity and placement of the reinforcement. If the pipes are tested for strength or absorption, inspection of the reinforcement shall be made on the pipe used for those tests, and in no case shall the total number of pipe cut open for inspection of reinforcement exceed the number to which the purchaser is entitled under the provisions of Sections 24 or 25.

34. Pipe shall be subject to rejection on account of failure to meet any of the specification requirements or on account of any of the following:

Causes for Rejection of Pipe—

(*a*) Fractures or cracks passing through the shell, except that an end crack that does not exceed the depth of the joint, or a fracture that at its deepest point does not exceed the depth of the joint nor extend more than ten per cent around the circumference shall not be considered cause for rejection unless these defects exist in more than five per cent of the pipe inspected.

(*b*) Defects which indicate imperfect mixing and molding.

(*c*) Exposure of the reinforcement when such exposure would indicate that the reinforcement is misplaced.

The Development of Specifications for Reinforced Concrete

By George J. Eyrick, Jr.*

The last five years have seen a phenomenal improvement in concrete, both in design, manufacture and construction; the last ten years, due to the untiring work of such men as Abrams, Young, Talbot and others too numerous to mention, we have seen the general acceptance of the various theories covering the design of concrete mixtures upon a rational basis, the use of "fineness modulus" for grading of concrete aggregates, the general recognition and use of the "water-cement" ratio, the development of the slump cone and flow table for checking workability and consistency, a general improvement in machinery for the manufacture and handling of concrete, and we are now approaching the point where standardization of tests of cement, aggregates and concrete is being realized.

Ten years ago the average architect and engineer in specifying a concrete mixture, would call for a 1:2:4 mix, paying little or no attention to quality of material used and no attention to the amount of water, and if he did specify a "clean, graded aggregate" he did not follow it up to see that he got what was specified. Water was commonly used to convey concrete so that it would flow along the forms under the reinforcing steel without spading.

The history of the "Joint Committee on Standard Specifications for Concrete and Reinforced Concrete" needs no mention in this paper, as every one is familiar with it, and all will give the Joint Committee full credit for the work it has done to bring about a well arranged and workable specification.

It was largely due to the work of the Joint Committee and the outstanding research men, whose work and findings were disseminated through the publications of the various societies and through the technical press, that we began seriously to consider a complete revision of our concrete specifications.

An investigation by our superintendents' department showed that the mason contractors paid little or no attention to their concrete work, and after some discussion it was decided that specifications and inspection should be made more rigid, with the result that a specification can now be developed for any strength concrete, and through proper supervision, a concrete of the desired strength can be obtained. All architects and engineers can obtain good concrete but it means that they must write an iron-bound specification and must have extremely rigid super-

* Specification Writer, Smith, Hinchman & Grylls, Engineers and Architects, Detroit.

CONC. COMPRESSION TESTS

FOR

NEW UNION TRUST BUILDING

Owners—New Union Building Co.

W. E. WOOD CO.
Bldg Const
Job 750

SMITH, HINCHMAN & GRYLLS
Archts. & Engrs.
File 5776

Location of Pour North 20th Floor 11 Bays

Mix 1-2½-4 Slump 7"

Fine Ag. 3.1 Coarse Ag. 6.1

Cement

DATES

Pour 5-11-28

Test 5-18-28

Test 6-8-28

Temp. Outside ______ Under Deck ______ Lab. ______

LAB No.	CYL AREA	7 DAYS		28 DAYS		SPEC. STR	WATER PER SACK	REMARKS
		TOTAL LOAD	LBS. per SQ "	TOTAL LOAD	LBS per SQ "			
2034	28.26	40550	1425		*2465		7½ Gal	
2035	"	38600	1360		*2406		"	
2036	"	40550	1425		*2465		"	
2034A	"			78100	2760	2000	"	
2035A	"			88000	3100	2000	"	
2036A	"			77100	2740	2000	"	

*Estimated 28 day strength.

REMARKS

All tests meet specifications

W. E. WOOD CO.
Per E. C. Reynolds

SMITH, HYNCHMAN AND GRYLLS
Per ______

FIG. 1—TYPE OF CYLINDER TEST REPORT, NEW UNION TRUST BUILDING, DETROIT.

vision to be sure that the specifications are followed, for the slightest let down means that the entire specification is voided as the material man, contractor and even the workman will take advantage of the let-down.

The architect and engineer are both interested in good concrete and take pride in turning over to the owner a structure that is attractive, substantially built and designed for his special use.

In considering specifications on concrete, plain and reinforced (this being the only part of the construction we are interested in), the owner for whom the work is done is interested in quality, speed and cost, the contractor and the construction superintendent are interested in cost, speed and quality. The sequence of these items is in accordance with their respective importance to the two parties to the contract. Both are interested in each, but to a different degree. The owner, who pays for the work desires a structure on which depreciation will be negligible, completed in as short a time as possible and at as low a cost as is consistent with quality. On the other hand, the contractor having given an estimate of the construction cost is vitally interested in keeping the cost below that figure. To him, speed means the ability to obtain his reward or profit at an earlier date, provided it can be done at equal cost with that obtained by a little slower progress. But even though the viewpoint of the two contracting parties are not exactly alike they are closely allied. Both are interested in quality, speed and cost.

Therefore, bearing the sequences in mind it is a problem for the architect to write his specifications so that both parties will be satisfied.

Prior to the adoption by the various member societies of the current Joint Committee Specification, our specifications were a rewrite of the "1914 Report" modified as necessary to meet the various job requirements. We will not run the risk of being tiresome by quoting the specifications, but will state that separate aggregates were not required nor was the amount of water specified to make a "medium wet mix," nor were any tests other than cement tests required. Cement tests were specified in accordance with the "Committee on Uniform Tests of Cement of the American Society of Civil Engineers."

As compared with the previously mentioned specification which we found wanting in many respects, we developed one which covered the concrete requirements for such buildings as the Greater Penobscot Building and the Union Trust Building. This revised specification required that "The Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete" shall govern all concrete work except where otherwise specified herein.

The grading of both fine and coarse aggregate are included as is also the amounts of water to be used per sack of cement. The fine aggregate was required to be inundated. We are requiring on all large building jobs that a field laboratory be installed in charge of a competent man, and equipped with the complete set of sieves, a sieve shaker and time switch, platform scale and breaking machine of approved type and of 200,000-lb. capacity.

The present specification calls for concrete cylinders "six for each pouring and class of concrete" and we are able to tell at all times how the concrete compares with the designed strengths. This method enables us to check the estimated 28-day strength against the actual 28-day break. (Figs. 1, 2 and 3 show the type of report received.)

It is impossible for us to check back very far to determine what benefits have been derived from this type of specification as complete records were not kept on work done under the older specifications, but a marked improvement has been seen in the quality of work since the new system has been in use.

FORM D1

PITTSBURGH TESTING LABORATORY
429 WAYNE ST., DETROIT MICH

REPORT OF CONCRETE CYLINDERS TESTED

Order No. De-2243
File No. 0304
Lab. No. 169-1937

Project Newcomb-Endicott Bldg.
At Woodward & Grand River Ave.-Detroit, Michigan
For Spencer, White & Prentis-Detroit, Michigan Customer's Order No.
Reported to " " " " " "
Date 3/3/28

MARK	DATE POURED	DATE TESTED	MIX	SLUMP	AGE DAYS	SECTIONAL AREA	TOTAL LOAD	POUNDS SQ. IN.	SPECIFIED STRENGTH 28 DAYS	MEETS SPEC.
297 - B	2/25	3/3	$1\text{-}2\frac{1}{4}$	2to4	7	28.27	113,000	3,997	5,000	--
297 - C	"	"	"	"	7	"	124,000	4,386	"	--
297 - T	"	"	"	"	7	"	106,000	3,750	"	--

297 = West Caisson Column #297

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ACS

MANAGER DETROIT DISTRICT

FIG. 2—TYPE OF CYLINDER TEST REPORT, NEWCOMB-ENDICOTT BUILDING, DETROIT.

As a piece of work is completed a graph is made up, such as is given in Fig. 4 for the Michigan Bell Telephone Co. building in Detroit, which shows the crushing strength of all test cylinders on the job. This graph is then printed and copies given to all parties interested. In this graph shown, it will be noticed that all breaks are well above the specified 2000-lb. line except the "Haydite" cylinders, and even with this aggregate, which was unfamiliar to the contractors organization, we had but one break below the specified strength. It will be well to remember in studying this chart that this operation was an addition of 10 stories to an existing 9-story building and the work was carried through the months of September to November, both inclusive. As will be seen as the temperature

dropped the strengths fell off slightly but in no case did they drop below the required strength. The results for the light weight aggregate are interesting, as we believe this job represents the first use of this material in this district for structural purposes.

It is impossible in many of our large cities to specify the water-cement ratio as their building departments claim that as yet they are not sure of the results. Therefore, there is yet much missionary work to be done and the architect and engineer can not be asked to do it all, and as a matter of fact they will not do it. Furthermore, when "controlled concrete" is specified, and the job is sent out for bids, some contractor will go to the owner with the claim that these "new fangled" ideas are going to cost him money and that the old methods are just as good and are cheaper. Hence the architect is hesitant in specifying something that will cost his client more money.

We specify separate aggregate for concrete, and invariably the contractor bidding on the work will include in his bid a very substantial amount as a credit if he is permitted to use "balanced" or combined aggregate. As all architects and engineers know it is annoying to have to explain to an owner why he is apparently throwing away money on some theoretical improvement in concrete especially where there are numberless usable concrete buildings in use which were built years ago. The owner can also ask why should he let his architect go to these extremes when many nationally and internationally known architects do not call for work of this kind, and to back up his contention he can cite the following, which covers the complete concrete specification, except for form work, placing, finishing, etc., for a prominent building in one of our large cities.

> "Concrete Material and Mixture:
>
> "Stone shall be sound broken limestone of sizes ranging from ¼ to ¾-in. evenly graded and mixed. If gravel is used, it must be clean, washed gravel, free from coke, and crushed if necessary so that it will pass through the same size screen as specified for broken stone.
>
> "Sand shall be clean, sharp, coarse torpedo sand, free from clay, loam or other foreign substances.
>
> "All concrete throughout shall be mixed by volumn in the proportions of 1 part cement, 2 parts sand and 4 parts gravel or cinders unless otherwise called for on the engineer's drawings."

Due to the results obtained from recent pieces of work in Detroit, the Department of Buildings and Safety Engineering of that city is now permitting the concrete specifications to be based upon the water-cement ratio. This type of specification is now being used upon the David Stott Building at the corner of Griswold and State Streets, Detroit, Donaldson & Meier, Architects.

The Albert A. Albrecht Co.
General Builders
Detroit

DAILY REPORT

Date Jan. 17th, 1928.

Lab. No. 52

Job Greater Penobscot Bldg. Architects Smith, Hinchman & Grylls

Owner Simon J. Murphy Co.

Location of Pour 17th & 28th Floors

Mix. 1: 2.8 : 3.8

Slump Slab 5", Columns 8"

W/C 1.13

Material Separate Aggregates

Course Aggregate Size 4 to 1" F.M. 6.5

Fine Aggregate " 0 to 4 F.M. 3.0

Cement

Remarks:

CYLINDERS TESTED

LAB No.	DATE POURED	DATE TESTED	X SECTION AREA	TOTAL LOAD	LBS. PER INS.	SPECIFIED STRENGTH PER INS	REMARKS
28-188	12-19-27	1-16-28	28.27	74000	2622	2000	17th fl. N.side Cols.
28-189	" " "	" " "	"	72000	2550	"	
28-190	" " "	" " "	"	79500	2820	"	17th fl. N.side Slab
28-191	" " "	" " "	"	62500	2214	"	
28-192	12-20-27	1-17-28	"	74000	2622	2000	17th fl. S.side Cols.
28-193	" " "	" " "	"	61000	2160	"	
28-194	" " "	" " "	"	10000	3540	"	17th fl. S.side Slab
28-195	" " "	" " "	"	91000	3222	"	
7-239	1-9-28	1-16-28	"	42000	1487 *2643		23rd fl. N.side Cols.
7-240	" " "	" " "	"	35000	1242 *2299		
7-241	" " "	" " "	"	43000	1523 *2643		23rd fl. N.side Slab.
7-242	" " "	" " "	"	40500	1433 *2523		
7-243	1-10-28	1-17-28	"	44000	1559 *2729		23rd fl. S.side Cols.
7-244	" " "	" " "	"	37000	1314 *[illegible]		
7-245	" " "	" " "	"	47500	1685 *2885		23rd fl. S.side Slab
7-246	1-10-28	1-17-28	28.27	37500	1332 *2412		23rd fl. " "

*Estimated 28 day strengths.

FIG. 3—TYPE OF CYLINDER TEST REPORT, GREATER PENOBSCOT BUILDING, DETROIT.

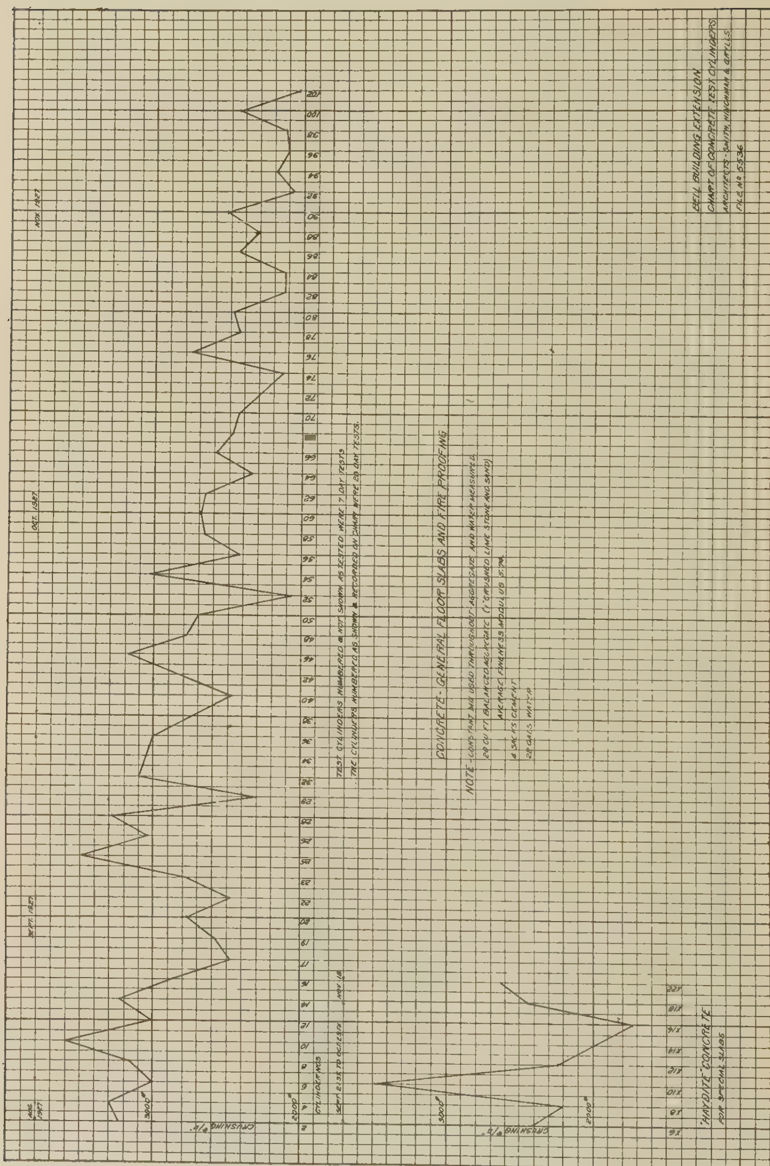

FIG. 4—GRAPH MADE UP AT END OF JOB SHOWING STRENGTH OF ALL CYLINDERS TESTED.

Albert Kahn, Inc., also of Detroit, usually includes the following "Alternate" in his concrete specifications which can be used wherever the cost difference is not too great or where it is desirable to use this form of specification in preference to the old type.

> "ALTERNATE: Contractor to state in tender amount to be added or deducted if concrete of a known strength instead of concrete of a known *mix* as above specified is used. In this case the following regulations shall be observed:
>
> "Contractor shall choose his own proportions of cement and aggregate such that accelerated tests (3 day and 7 day tests) show that the desired strengths will be obtained. He may use a weighing scale, or an inundator with separated aggregates, or other method approved by the Department of Buildings which will insure uniformity in the predetermined strength of the concrete.
>
> "All concrete for footings, walls, concrete stairs, floor slabs and fireproofing of steel work shall have a minimum strength of 2000 lb. per sq. in. at 28 days.
>
> "2000-lb. concrete shall be made using 7 gal. of water per bag of cement. The water specified includes any and all water already contained in the sand or gravel and only the difference already contained in the aggregate and the quantity specified shall be added. Concrete shall be mixed 1 full minute after all materials are in the mixer; the time being gauged by watch or clock, not by counting mixer revolutions. The drum shall have a peripheral speed of about 200 ft. per min. and shall be kept adjusted to that speed. No defective mixer will be allowed on the job.
>
> "The consistency of footing concrete shall be given by 5-in. slump, obtained by slump concrete measurements. The consistency of wall concrete shall be that given by a 6-in. slump, and the consistency of floor, stair and fireproofing concrete, that given by a 7-in. slump."

This paper is primarily to show the development of specifications on concrete, plain and reinforced, in one office, and reference is made to two other offices to show that progress is being made in the right direction. There is a vast amount of room for development and missionary work to put across the basic principles of a good concrete. Much must be done to show the contractor that *good* concrete can be made at no greater expense than in the old system. Material men must be shown that it is to their advantage to furnish the material as specified and not as they desire, although their way would be the easiest for them. A study of local aggregates should be made and the results broadcast to all persons interested. The subject of cements should also be studied very carefully as one cement is not suitable for all purposes and it is the writer's belief that cements of various characteristics should be developed as is now the case in Germany and other European countries.

Mr. Meacham. James A. Meacham—It might be of interest to Mr. Eyrick and others to remark in passing that the H. K. Ferguson Company is now employing a strictly water control specification for its entire concrete production. This involves work over a very considerable part of the United States and the Orient and the use of a range of concrete aggregates quite beyond the limits ordinarily expected when work is being conducted within the limits of a certain geological area.

The use of this type of specification permits full advantage to be taken of the possibilities of aggregates available at any given job site, and likewise affords a flexibility which is of the greatest value in those cases where the local supply or even distant sources do not offer materials of recognized quality. In other words, the proper measures for overcoming deficiencies in the aggregate may be taken with the least amount of friction and without jeopardizing the strength of the concrete.

Mr. Bischoff. J. M. Bischoff—The Department of Buildings of the City of Detroit for the last three years has allowed the use of the water-cement ratio to be optional with the contractor. A good many did not want to use it and the same may be said of the architect. However, the department, since I have been in it, has never turned it down and does not intend to.

Mr. Lindau. Chairman Lindau—Has their experience been that there was an increase in cost by the use of the water-cement ratio?

Mr. Bischoff. J. M. Bischoff—Not that I know of. I do not think so.

Mr. Hart. W. E. Hart—In our experience in the last few years we have found that in almost every case where the water-cement ratio form of specification was being used there was a small saving, a saving sufficient to pay the salary and expense of hiring a special engineer.

Mr. Dutton. Earl Dutton—I would like to ask Mr. Hart if he is not relying on the size of the job. How large must a job be in cu. yd. of material to balance off that cost? I do not think in small jobs of 1000 or 2000 cu. yd. that the use of a water-ratio specification would be warranted.

Mr. Hart. W. E. Hart—I do not know that I can answer that definitely, but it would take a job from 2000 cu. yd. up.

Mr. Ahlers. J. G. Ahlers—I think I can answer that. We have, in the last four or five years, run all jobs with water control, even as low as 500 cu. yd., and in every instance have shown a saving.

Mr. Lindau. Chairman Lindau—Including the inspection required?

Mr. Ahlers. J. G. Ahlers—Yes, sir.

H. C. McCall—Both of the gentlemen discussing this matter have failed to point out the additional factor that almost always the concrete will be of higher quality and much more uniform. Mr. McCall.

A. S. Douglass—I would like to add to that last remark that we have been forced to specify an arbitrary low limit of cement, not because we cannot get 2000-lb. concrete easily, but because we are afraid the amount of cement that will give us 2000-lb. concrete will make porous concrete that will not weather. Mr. Douglass.

Purchasing Centrally Mixed Concrete

By P. J. Freeman*

The development of central mixing plants during the past few years has necessitated a revision of specifications as well as of general ideas concerning mixing and handling concrete. In 1925 there were about 25 such plants and today there are at least 100 concerns making and selling ready-mixed concrete.

One of the first questions encountered by the early purchasers of centrally mixed concrete was the effect of hauling and the determination of a maximum time allowable between mixing and placing the concrete.

The writer is indebted to Prof. F. E. Gieseke for the information given in a paper before the American Society for Testing Materials in 1920 on the "Effect of Rodding Concrete." The method used by Professor Gieseke was to agitate fresh concrete by means of rods so that the air was removed and the time of final setting delayed. The test data showed that the strength of concrete was not injured by disturbing the initial set and that a very decided increase in strength could be obtained by agitating the concrete long after the degree of hardening known as the initial set had taken place.

When the writer was first called upon to approve the use of about 28,000 cu. yd. of centrally mixed concrete for the land piers of the Liberty Bridge, being built by Allegheny County in 1925, there were no data at hand to show the effect of hauling concrete and no time to make such tests. The decision to use ready-mixed concrete was based largely upon the conclusions of Professor Gieseke, for the writer believes that they apply equally well to properly proportioned concrete which is being hauled as to the methods used by Professor Gieseke in his laboratory.

The piers were to be built in the Baltimore & Ohio Railroad yards and there was scant room for operating ordinary mixing plants in winter weather. The fact that all materials could be easily heated at the central plant and accurately proportioned, led us to give serious consideration to the first request to use centrally mixed concrete.

The length of the haul was about 2½ miles and the standard specifications required all concrete to be placed inside of 45 min. after mixing. The practice of having our plant inspectors give a ticket to the truck driver to be delivered to the inspector at the job was started. This practice is still continued and although the time limit of 45 min. may not be necessary, conditions are such that it is not necessary to exceed this

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time. The tickets are in duplicate and carry the particular job number, proportions used, amount of water, date and exact time of emptying the mixture into the truck, stamped by a time clock.

Permission to use centrally mixed concrete was therefore granted with the further stipulation that 1-cu. yd. buckets be used having drop-bottoms so that the concrete could be readily placed in the forms without additional handling. It required only a short time to enter the railroad yard and lift the buckets from the trucks with a hoist and deposit the concrete in the forms, and the process did not in any way interfere with the railroad.

Tests were made on 6 x 12-in. compression specimens at the mixing plant and at the piers, but no falling off in strength was encountered and the practice of making specimens at the mixing plant was adopted on account of the greater convenience for curing in cold weather.

Similar tests have been made at various times on other jobs, but nothing has been found to indicate the lowering of concrete strength due to hauling. If the concrete is not segregated when it arrives at its destination, the quality is not impaired.

The results of tests given in *Public Roads*, September, 1928, indicate that the quality of concrete is not injured by hauls up to 25 miles in distance or 3 hours duration, provided the material is delivered in a workable condition and will not need additional water to facilitate its handling.

After the very satisfactory experience with the land piers of the Liberty Bridge, centrally mixed concrete was used for various purposes in Allegheny County. The most spectacular structure was a concrete slope wall 90 ft. high which required approximately 10,000 cu. yd. of concrete. The mix was 1:2:4 gravel concrete and the material was handled in 1-cu. yd. buckets and placed with a crane. The length of haul was about 2 miles. The materials were workable when they arrived on the job and it was possible to place the concrete in a rather narrow section without an undue amount of honeycombing. The exposed surface of the slope wall is approximately 30,000 sq. ft. and it was not necessary to retouch more than 20 sq. ft. of the entire surface.

Concrete for the base course of a small bridge floor was hauled to a suburb of Pittsburgh, a distance of approximately 9 miles, with only partial success. The mix and materials were the same as for the slope wall, but the concrete was hauled in ordinary dump trucks. The first few truck loads in the morning would arrive in a very satisfactory condition on account of the fact that they were not delayed by traffic. Later in the morning when traffic began to delay the trucks the long haul over very rough streets produced a segregated concrete which was not satisfactory. It was necessary to re-mix the concrete with shovels before it could be placed, but the addition of water was not permitted.

These three instances represent typical conditions which have been encountered with centrally mixed concrete involving the placing of more than 50,000 cu. yd. Our contracts are with a general contractor who is responsible to the county for all sub-contractors including a central mix-

ing company, and the mixing and placing of concrete must comply with the printed standards of construction whether the concrete is made on the job or at a central plant. These specifications cover all details from the filing of a bond to the final acceptance of the structure.

The use of centrally mixed concrete for any particular job is therefore subject to the approval of the engineers of the Department of Public Works before the job can start.

The central mixing plant equipment may be better than that which is available in the field and for that reason we like to use concrete from a ready mixing plant whenever it is possible. If the area of a pier is so great that it requires a large amount of concrete per hour to keep from disturbing that which has partially set, it may be necessary to require the general contractor to furnish one or more concrete mixers to supply concrete in addition to that coming from a central mixing plant.

These details must be agreed upon before the general contractor receives permission to use concrete from a central mixing plant. With the increased capacity which the larger units are developing and the improvements in methods of hauling and placing concrete, it may be possible for us to remove some of these restrictions in the future.

If the concrete is to be purchased directly from the central mixing company, then there are a number of considerations which must be taken into account that do not concern a municipality which is having work done by a general contractor under standard specifications. It is the duty of the engineer responsible for the work to see that his purchase orders or specifications are so drawn that the interests of the owner in whose structure the concrete is being used are properly safeguarded.

The size and nature of the work will, of course, govern the extent of these requirements. A few general suggestions will be given which should be considered in preparing such a purchase order for centrally mixed concrete to be used in a rather important construction project; more items can be added for larger work and some of these eliminated for lesser structures:

(a) Investigate all mixing plants available and note the general nature of the equipment and methods of operation. Inspect the measuring devices and observe the quality of materials available.

(b) If the proportioning devices permit the weighing of all cement, sand and coarse aggregate, the basic mix (preferably determined in advance from tests) should be stated giving definite weights for each material, from a given source

If the water-cement ratio is to be used for proportioning, the desired limits should be placed on the water and also on the aggregates.

(c) State the minimum time of mixing which will be permitted.

(d) Fix the minimum temperature at which concrete shall be delivered on the job (70 deg. F.).

(e) State unit price for addition or deduction of cement, sand or coarse aggregate. Make provision for changing the mix if it is found desirable.

(f) Establish unit price for the addition of any desired admixture.

(g) State definite basis of payment for mixed concrete delivered on the job. If all materials are weighed, it is preferable to base this price on the actual weight of mixed concrete delivered, rather than on a cubic yard basis.

(h) Provide for the engineer's inspection at the central mixing plant and for tickets to accompany each delivery truck.

(i) Provide for reasonably uniform delivery of so many yards per hour (60 yd. per hr. for an ordinary job is a good rate), depending on the size of the job, number of hoists or other equipment used to handle the concrete when it arrives. Have an understanding as to the type of truck which will be used for hauling the concrete. In some cases it may be necessary to require special equipment or trucks to handle the concrete when it arrives on the job. Some central mixing plants can furnish rotating drums or various types of trucks which will agitate the concrete for long hauls.

(j) Provide for testing a definite number of concrete specimens by a laboratory to be selected by the engineer, but at the expense of the central mixing company. The number of specimens should be governed by the quantity of concrete placed or in some way which cannot be misunderstood. This feature frequently causes ill-feeling because the central mixing company may feel that the engineer is selecting more specimens than the price given warrants and a definite understanding should be had before the work starts.

(k) Provide for cutting off delivery and disposal (to other work) of concrete en route in case of accidents on the job or for a hold-up in construction.

(l) Make provisions for cancellation of the contract in case the engineer finds cause for doing so. Provide for rejection of loads received in bad condition on account of segregation, excessive water or extreme delay in delivery. In the cases that the work is handicapped by the failure of the central mixing company to make delivery according to the arrangement, provision should be made to reimburse the owner or general contractor for the expense of installing his own mixing plant.

(m) Have a definite understanding regarding charges for delivery before or after regular working hours or on Saturday afternoon and Sundays. There should either be no charge or a fixed sum for such over-time work.

(n) Arrange for guarantee against labor trouble in case the mixing plant uses non-union labor in a closed district.

(o) Provide for surety bond if the volume of work warrants.

Mr. Foster.

A. Foster—Mr. Freeman states that there are today at least 100 concerns marketing concrete from central mixing plants. It therefore behooves the engineer who has concrete structures to design and inspect to acquaint himself with this development which has upset some of the old fixed ideas as to the manner in which cement, sand, and gravel or stone should be handled to produce good concrete.

In 1921 the U. S. Bureau of Roads conducted tests on concrete hauled in a truck from which from time to time samples were taken. If memory serves, no decrease in strength was found until after the seventh hour. Workability, however, had disappeared before this time.

Those who merchandised concrete met on all sides statements by engineers, that the whole idea of central-mixed concrete was wrong and that all work constructed with concrete from a central mixing plant, delivered by truck, would prove faulty. Many tests, in a period of eight years, proved that the so-called faulty concrete was stronger upon delivery than when mixed at the plant. Concrete from a central plant proved by tests to be as strong and in many cases stronger than that mixed at the site of the work and held so many advantages, particularly for street and road work, that authorities let down the bars and within certain limits of time and distance permitted its use.

Undoubtedly the vast increase in the number of central mixing plants is, in a great measure, due to their distinct advantage to the engineer and the builder. One of the greatest troubles with the usual concrete plant on building or road work is the high rate of turnover of the operating force. A mixer crew may go from job to job in some cases, but particularly with the small contractor the concrete force comes and goes. This produces inefficiency and is one of the weaknesses in the manufacture of concrete.

At the central mixing plant all possible labor-saving devices are used, the engineering is much better than that found around the average mixing plant, and the personnel, by reason of this fact, is small in number, of a higher type, and kept on from year to year. The consequence is a uniform product manufactured by and under the supervision of competent, high-grade skilled workmen. Control of ingredients by weighing and observance of water-cement ratio all become simplified at a central plant. Inspection can be made at one plant for many jobs; especially is this true for street work. All variables are reduced to a practical minimum.

In Philadelphia the first commercial plant was constructed in the

spring of 1921. Concrete was hauled in regular dump bodies with more or less tight tailgates. Concrete in the usual dump body proved harsh and lacked workability. This was due to the water lost through leaky tailgates. This lost water was discolored it is true, but quite a bit of the color was due to the silt in the sand. Whether the difference between the specific gravity of the cement and water let the water escape, or whether the cement with the water needed for its hydration was in a pasty state, the fact remains that but little cement was lost with the water which dripped from trucks transporting concrete.

A body was next tried which did not have a tailgate but a bottom which sloped up near the back of the truck. This body was capable of being raised to nearly 80 deg. to the horizontal and the sloping rear retarded the concrete so that there would not be any surge. An increase in workability was noted at once.

With an open top truck there is a certain amount of evaporation of water varying with temperature and atmospheric conditions. Whether or not there is a reduction in strength in an hour or thereabouts between the mixer and the job is a disputable question, but it is a fact that the reduction in the water-cement ratio and the slump between mixer and job nearly always produces strength higher at the job than at the mixer.

Tests have been made on concrete delivered by closed revolving truck bodies which indicate a reduction between mixer and job amounting to 10 per cent of the compressive strength. Is this due to the closed truck body or to some other condition? Many are the types of special bodies designed to agitate concrete while in transit. Strong sales forces are at work to put before the engineer the absolute necessity of agitation.

Agitation may or may not be beneficial to concrete in transit. It certainly will produce a more workable concrete than when no type of agitation is used. Where fixed mixes such as, 1:3:6, 1:2:4, etc., are strictly adhered to, segregation may result in the delivery of concrete even with the use of admixtures, especially if the mix is one having a high slump. In these cases agitating bodies are very beneficial. However, with properly designed mixes for purposes other than use in thin highly reinforced walls or slabs, workable concrete can be delivered within one hour and without segregation by means of tight bodies and without agitation. In the city of Philadelphia the Bureau of Highways has placed a time limit of 45 min. for the maximum haul. A truck can take a load 9 miles in this time, so that with the location of the mixing plants in Philadelphia as centers, the whole city can be served in the time allowed. The Bureau of Buildings specifies a period of 1 hr. between mixing and placing. This means that all central Philadelphia can be served even with due allowance for traffic jams.

The time factor, in the light of recent tests of the Portland Cement Association, should not be thought of as vital up to 3 hr., except as it results in decreased workability. In the limits specified in Philadelphia we believe that workable concrete with a slump of 1 to 5 in. can be delivered in open-top tight bodies without agitation and serve street

work, foundations, underpinning, foundation walls, etc. For thin slabs, walls, etc., where the concrete must have a slump of 6 to 8 in., trucks having some means of agitating the concrete probably will produce best results.

Where standard mixes are specified, if central mixed concrete is to be used, it will be found advantageous to resort to the use of some one of the more common admixtures. In Philadelphia, hydrated lime has been used in some cases and celite in others. Hydrated lime when added to the leaner mixes seems to produce workability and strength. Even when a higher sand content is used than the old conventional 1:2 proportion the fattening of the mix by an admixture seems to produce good results. Within reason it is well to run up the fines (50 mesh) in the concrete sand beyond the limits used in some states for concrete roads. This increase in 50-mesh fines together with the admixture, increases workability and decreases the possibility of segregation.

In regard to the operation of a central mixing plant to serve the needs of the engineer in obtaining the best results from concrete designed for various types of construction, it would seem well to have the plant use the proportion of sand to gravel, stone or other coarse aggregate which will produce the densest concrete. This will vary from 35–65 per cent to 45–55 per cent depending upon locality, grading, etc. With the relation of the aggregate fixed, a varying cement content with a desired water-cement ratio will produce the results required. It is better in some cases for the central mixing plant to keep the weights (where weighing is the method of measurement) of sand and gravel constant between the mixes of, say, 1:3:6 and 1:2:4, varying the cement and water to suit. The increased volume of concrete in the richer mixes can be determined and the billing made thereon. With a plant serving several jobs of varying mix there is a certain amount of time lost in changing weights.

One link in the chain between mixer and job which is of vital importance is what might be termed a contact or trouble man. He visits the job from time to time, sees how the work is progressing and the concrete is being handled. If deliveries are more than can be properly handled, or vice versa, he corrects same. Near the end of the day he endeavors to prevent over shipment. His services are particularly required on the smaller jobs where supervision is apt to be negligible.

Traffic through the heart of most American cities is so congested as to require 100 per cent of the highway curb to curb. Where this is not so, that part not required for moving vehicles is devoted to parking space. Particularly in the older towns along the Atlantic seaboard, sidewalks are none too wide and streets are narrow. Certainly the authorities of these towns will not become more lenient in their rules governing the use of streets and sidewalks for the storage of building materials. The basement of the building becomes the storage space for materials. The stoppage of traffic while deliveries of materials are made produces traffic jams. On the streets themselves where repairing is contemplated the tie-up of traffic must be kept to a minimum and no street

intersections can be used for storing materials. All these factors trend to make the central concrete mixing plant a vital factor in the road and building business and codes or regulations not contemplating the use of concrete from such a plant cannot be viewed from the standpoint of economy or good engineering practice.

W. E. HART (*By Letter*)—Mr. Freeman has presented a logical and well thought out plan for specifications for a project where centrally mixed concrete is to be used. The general use of this type of concrete not only must be reckoned with in the near future, but it has already become a factor in building operations. There are at least 100 central mixing plants in the country. As a rule the operation of these plants is in the hands of competent men, and the class of concrete is equally as satisfactory as that made on construction jobs under the old method of mixing concrete. Specifications for plant-mixed concrete should be so drawn as to protect first the contractor and owner, and second the plant operator. **Mr. Hart.**

Central mixing plants offer an opportunity to deliver concrete to a job designed for a definite strength and purpose. The operation of mixing the concrete is invested in one man whose responsibility is more than that of making a good concrete. He must invariably answer at the end of the year to a board of directors, with the result that good-will and a reasonable profit must go hand-in-hand with the delivery of a good product.

Engineers should also consider the opportunity of designing their concrete for its service or exposure in the structure. The central plant, together with the "tag-delivery" system proposed by Mr. Freeman, permits the making of concrete for a definite part or member of the building. Outside columns and concrete exposed to weathering may be mixed in the ratio of not more than 6 to 6½ gal. of water per bag of cement while interior members may be mixed in the ratio of not more than 7 to 7½ gal. per bag of cement. In other words, the arrangement of equipment in the central mixing plant gives a flexibility for the adjustments or changes in mixes necessitated by the changing conditions on a job. Water and aggregate control can be more closely watched under these conditions than on the small projects where the contractor cannot afford to set up a large proportioning plant.

Mr. Freeman has drawn a number of conclusions in his paper. He has pointed out the necessity of watching certain items which are particularly referred to in the paper. I am offering some discussion on a few of these items:

(*a*) Specifications may well require that plants supplying centrally-mixed concrete be equipped to a definite minimum standard. Besides this, the specifications should also require that the plant be under the supervision of a competent man. By this means only is the engineer able to safeguard his specifications and deliver to the job the proper kind of concrete.

(*b*) The engineer should plot his job curve for concrete strengths the same as recommended on a regular construction project. Arbitrary mixes

should not be encouraged where the plants are equipped to produce concrete under the water-cement ratio form of specifications. The contractor should be given an opportunity to take full advantage of any saving due to the increased efficiency in his building operation. In other words, should the contractor, by increased efficiency, be able to place a concrete of a stiffer consistency than that specified, he should be given any advantage or saving thus gained through his contract with the central plant owner.

(*c*) In many cases the time of mixing concrete in a central mixing plant can be increased because of uncertain factors in transportation. However, the minimum requirements of the specifications should not be violated in case prompt delivery is being delayed by unforeseen conditions.

(*d*) The specifications should also set up definite requirements as to the production of concrete in the mixing plant and in transportation for cold weather work. Central mixing plant owners should be encouraged to build into their bins a system of steam coils for heating materials. Each truck-load of concrete should be required to be covered with a tight fitting tarpaulin to prevent the dissipation of heat while in transit. The concrete should have a minimum temperature of 60 deg. when placed in the forms.

(*i*) The provisions set up by Mr. Freeman for the uniformity of delivery of concrete to the project are exceptionally good. The cycle of operations on the job must be maintained by the plant. Contractors depend upon this cycle in their building operations to keep the job going smoothly and maintain their organization. The specifications should also cover the class of truck body to be used for the delivery of the concrete. The type of body will depend largely upon the distance the concrete is to be transported and upon the class of traffic or congestion through which the truck must pass. One type of body may serve well for the short haul while an entirely different type must be used for the 10 to 20-mile haul. It is therefore suggested that on the long haul some form of agitator truck be required. Such trucks have been giving excellent satisfaction in delivering concrete of various consistencies without segregation.

(*j*) Methods of making tests on concrete should be as recommended by the American Society for Testing Materials. A number of tests for a given yardage of concrete are specified in detail in the Tentative Report of Committee E-1 of this Institute. Both of these items should become a definite part of any engineering specification, particularly where centrally-mixed concrete is involved. One item that is of major importance is the place at which test specimens are to be made. In other words, it is best to take the test specimens on the job and under job conditions rather than take the specimens at the plant. The reference in the specifications to the two standards above mentioned will eliminate the possible controversy mentioned by Mr. Freeman.

(*k*) A provision in the specifications for the cutting off of delivery of concrete to the job should be so worded as to protect both the contractor and the owner, and the central plant manufacturer. Accidents

may occur at either the plant or the job, provision for which should be made in the specifications.

As time goes on and centrally-mixed concrete becomes more generally used it will be necessary to develop a type of specification that will be peculiarly fitted to this method of operation. Under no conditions should the engineer lose sight of the fact that there is equal responsibility on both sides of the specifications. Certain obligations must be set up whereby the central plant owner is bound and held responsible. At the same time the plant owner's interest must not be overlooked and he should not be ruled out of existence by extreme provisions in the specifications. There is a large field for the sale of ready-mixed concrete and it can best be encouraged by proper specifications. I would therefore urge that this subject be studied by the Institute and a specification returned that would tend to encourage the use of centrally-mixed concrete on public and private work.

H. F. THOMSON—May I make one comment on Mr. Hart's suggestion? The expressions, "ready-mixed concrete" and "central-mixed concrete" have both been used. My suggestion is that in considering Mr. Hart's suggestion of specifications, that the idea of ready-mixed concrete be applied to both central-mixed and truck-mixed operations. There has been a tendency to confine the term "ready-mixed concrete" to the product of central mixing plants. I think it is pretty well recognized that truck mixing operations are also going to be of importance and they should be considered in connection with a central-mixed specifications such as Mr. Hart suggested. **Mr. Thomson.**

Confusion of Specifications for Aggregates

By L. E. Williams*

Concrete aggregate producers have viewed with longing, if not envy, the results which are being obtained in many industries in the matter of standardization. The discussion at the 1928 convention of the lack of standardization in the concrete stone field[1] led the writer to believe that a presentation of a somewhat similar problem relating to concrete aggregates would be of interest, and might lead to beneficial results.

It is not the intent of this paper to offer any solution of the problem in the form of tentative standardization of specifications, but rather to show the diversity of aggregates called for in the territory adjacent to Detroit and the resultant effect on materials produced in an attempt to meet the varied requirements.

For several years the Concrete Committee of the Detroit Engineering Society has worked on this problem but up to the present the results have not been very encouraging. It is hoped that eventually the continued effort will bring about a reduction in the diversity of specification at present in use in this district. In collecting the data herein presented the files of this concrete committee have been used to a considerable extent, as well as the suggestions of several of the committee members.

From the standpoint of specifications there are four major difficulties encountered by aggregate producers. They are (1) the multiplicity of sizes specified, (2) the fact that both sieves (square openings) and screens (round openings) are used in specifications, (3) the great diversity of specifications for aggregates used in the same type of construction, and (4) the indefiniteness of certain clauses used.

Fine Aggregates—There are between 10 and 15 specifications for fine aggregates used in concrete construction in this district, not including materials used in the manufacture of concrete products. However, the major portion of this output falls under the eight specifications listed in Table 1. In Table 2 are shown mechanical analyses of 5 different typical concrete sands. These samples were taken from material turned out in normal commercial production. In Table 3 these 5 samples are classified as to their acceptability under the specifications given in Table 1, and in Table 4 are given the reasons for rejections. You will note that none of the samples are acceptable under specification No. 3 on account of too great retention of the 50-mesh sieve.

* Director and Engineer, State Concrete Materials Co., Detroit.

[1] "Specifications for Concrete Stone," by C. Van de Bogart, *Proceedings*, A.C.I. 1928, p. 348.

In addition to the 10 to 15 fine aggregate specifications for concrete construction which have to be furnished from time to time there is a

TABLE 1—FINE AGGREGATE SPECIFICATIONS
Per Cent Passing Sieves and Screens

	Paving					Tunnel	Structural	
	1	2	3	4	5	6[1]	7	8
⅜-in. sieve	....	100	....	....	....	....	....	100
⅜-in. screen	....	....	....	100	....	....	....	....
¼-in. screen	95–100	....	....	90–100	100	....	....	....
¼-in. mesh sieve	....	....	100	....	....	....	....	....
4-mesh sieve	....	95–100	....	....	....	95–100	85–100	96–100
8-mesh sieve	....	....	....	....	....	....	65–90	68–88
10-mesh sieve	....	....	....	60–90	....	....	....	....
14-mesh sieve	....	....	....	....	....	....	....	48–68
20-mesh sieve	35–65	25–65	....	25–65	35–75	....	....	....
48-mesh sieve	....	....	....	....	....	....	....	6–20
50-mesh sieve	0–25	5–20	15–25	7–25	0–25	....	10–30	....
100-mesh sieve	0–3	0–5	0–5	0–5	0–5	0–5	....	0–5

[1] Uniformly graded, fineness modulus 2.75 to 3.25.

TABLE 2—ANALYSIS OF TYPICAL COMMERCIAL FINE AGGREGATES
Total Percentages

	Samples				
	A	B	C	D	E
Retained on ⅜-in. sieve	0.	0.	0.	0.	0.
Retained on 4-mesh sieve	3.6	9.4	4.5	2.5	0.
Retained on 8-mesh sieve	13.7	21.4	15.7	16.1	2.9[1]
Retained on 14-mesh sieve	44.4	40.9	32.8	44.1	28.3
Retained on 28-mesh sieve	69.2	59.9	61.6	70.7	58.2
Retained on 48-mesh sieve	91.0	88.3	88.7	88.2	96.8
Retained on 100-mesh sieve	98 1	98.5	97.8	99.2	99.7
Passing 100-mesh sieve	1.9	1.5	2.2	0.8	0.3
Fineness modulus	3.20	3 18	3.01	3.21	2.86

[1] 100 per cent passed ¼-in. screen.

TABLE 3—CLASSIFICATION OF AGGREGATES SHOWN IN TABLE 2 UNDER SPECIFICATION IN TABLE 1

Sand	Acceptable for	Not Acceptable for
A	1, 2, 4, 6, 8	3, 5, 7
B	4, 7	1, 2, 3, 5, 6, 8
C	1, 2, 4, 6, 7	3, 5, 8
D	1, 2, 4, 6, 7, 8	3, 5
E	1, 5, 6	2, 3, 4, 7, 8

continual demand for materials for concrete products, as well as sands for plastering, brick work, floor finish, etc. There are approximately 12 different kinds of material required for concrete products alone.

In general there are no definite specifications for the miscellaneous sands mentioned, nor for the majority of concrete products aggregates. The following terms are used, referring supposedly to the mechanical grading of the sands: "flat," "sharp," "fifty-fifty," "granite" and "torpedo." There is also the following nomenclature based on the use: "masons," "brick," "plaster," "concrete" and "floor finish." All of these terms are, to say the least, indefinite. Flat sand refers to the finer grades and sharp sand means the coarser grades, while fifty-fifty sand is supposedly a combination of these two. The terms "flat" and "sharp" have practically no reference to the smoothness nor the angularity of the grains but apply only to the coarseness of the material. Granite sand is a name used for certain floor finish sands while torpedo sand usually

TABLE 4—REASONS FOR REJECTION OF SAMPLES SHOWN IN TABLE 2

Sample	Specification No.	Reasons for Rejection
A	3	Retention on 4-mesh sieve and 50-mesh sieve too great
	5	Retention on 4-mesh sieve too great
	7	Retention on 50-mesh sieve too great
B	1	Retention on ¼-in. screen too great
	2	Retention on 4-mesh sieve too great
	3	Retention on ¼-in mesh sieve and 50-mesh sieve too great.
	5	Retention on ¼-in. screen too great
	6	Retention on 4-mesh sieve too great
	8	Retention on 4-mesh sieve too great
C	3	Retention on ¼-in. mesh sieve and 50-mesh sieve too great
	5	Retention on ¼-in. screen too great
	8	Retention on 4-mesh sieve too great
D	3	Retention on 50-mesh sieve too great
	5	Retention on ¼-in. screen too great
E	2	Retention on 50-mesh sieve too great
	3	Retention on 50-mesh sieve too great
	4	Retention on 50-mesh sieve too great
	7	Retention on 50-mesh sieve too great
	8	Retention on 14-mesh sieve and 48-mesh sieve too grea

refers to a particularly coarse sand fairly free from fine particles. The terms based on the use are self-explanatory as to the purpose intended but extremely variable as to the quality desired.

To further complicate matters the personal element enters. Mr. A's flat sand may be Mr. B's sharp sand, while Mr. C's concrete sand may be Mr. D's torpedo sand. In addition to this, in the case of customers who are retail dealers, their ideas as to what they want under certain designations often vary from time to time depending upon the demands of the retail trade.

About the only safe way to handle the requirements of the average customer for sands not covered by a definite specification is to have him select a sample that will fill his needs, then proceed to analyze the sample and ship accordingly. Even then visual inspection by the customer may result in arguments.

Undoubtedly many of the materials above referred to do not fall within the field of this body, but their demand does greatly complicate production in plants whose major output is for concrete work.

Coarse Aggregates—Coarse aggregates may be broadly classified as to their use under paving, tunnels and sewers, and structural. The last classification covering of course a very broad variation as to type of work, but a much more limited variation as to aggregate requirements. In Tables 5, 6 and 7 are given the 6 major specifications, from the standpoint of volume of material used in this territory, under each of these classifications. There are in addition to these at least as many more, of greater or less diversity, which have to be filled from time to time, but in general not in quantities comparable with those given in the tables.

Referring to the paving aggregates, Table 5, if you disregard specification No. 13 it is possible to produce an aggregate that will fill all of the other specifications, but the band common to these is so extremely narrow that it is impossible to produce commercially in any quantities. The differences in these specifications are not predicated on either differences in local conditions nor design, but represent differences of opinion as to the material required for the same type of work under common conditions.

The tunnel and sewer specifications given in Table 6 should be looked at somewhat differently as the question of type of construction must be taken into consideration; nevertheless it is doubtful if even this would warrant the existing variations.

When we consider coarse aggregates for structural concrete many factors must be recognized which are to a considerable extent absent from the types of construction previously considered. Conditions of reinforcement, thickness of members, workability, etc., must be given careful consideration. Table 7 shows considerably less than one-half of the different structural coarse-aggregate specifications at present in use here. It would be impossible to draw any fair comparison of these specifications without considering all of the factors involved. Those given have been taken from specifications for comparatively heavy construction and do not cover the many in use for close reinforcement, thin slabs, fireproofing, etc., which make up a rather voluminous collection by themselves.

A survey of the specifications in use during the current season shows "100 per cent passing" called for in the following sizes:

2½-in. screen	2-in. sieve
2¼-in. "	1¾-in. "
2-in. "	1½-in. "
1½-in. "	1-in. "
1¼-in. "	¾-in. "
1-in. "	⅜-in. "
¾-in. "	4-mesh sieve
⅜-in. "	8 " "
¼-in. "	

TABLE 5—COARSE AGGREGATE SPECIFICATION FOR PAVING
Per Cent Passing Sieves and Screens

	9	10	11	12	13	14
2½-in. screen......	95–100	......	......	100	......	100
2-in. sieve..........	......	100	......	......	......	......
2-in. screen........	......	......	100	90–100	100	......
1½-in. sieve........	......	85–95	......	......	......	......
1½-in. screen......	50–85	......	......	......	......	70–100
1¼-in. screen......	......	......	......	......	90–98	......
1-in. screen.........	......	......	60–80	30–70	......	......
¾-in. sieve.........	......	45–55	......	......	......	......
¾-in. screen.......	......	......	......	......	......	30–60
½-in. screen.......	......	......	......	10–40	not less than 10	......
¼-in. screen.......	0–10	......	0–10	0–10	0–10	0–4
4-mesh sieve.......	......	0–10	......	......	......	......
8-mesh sieve.......	......	......	......	0–5	......	......

TABLE 6—SPECIFICATION FOR COARSE AGGREGATE FOR TUNNELS AND SEWERS
Per Cent Passing Sieves and Screens

	15[1]	16	17	18	19	20
2-in. sieve...........	95–100	......	......	......	......	......
1½-in. screen.........	......	95–100	100	100	100	......
1-in. sieve............	......	......	......	25–75	......	......
1-in. screen...........	......	......	......	......	75–90	100
¾-in. screen..........	......	40–75	......	......	35–75	75–90
½-in. screen..........	......	......	......	......	......	35–75
¼-in. screen..........	......	0–15	0	......	......	......
4-mesh sieve.........	0–5	......	......	0–15	5–35	5–35
8-mesh sieve.........	......	......	......	0–5	0	0
⅛-in. screen..........	......	0–5	......	......	......	......

[1] Uniformly graded, fineness modulus 6.90 to 7.60.

TABLE 7—COARSE AGGREGATE SPECIFICATIONS FOR STRUCTURAL WORK
Per Cent Passing Sieves and Screens

	21	22	23	24	25	26
2-in. sieve..........	......	......	95–100	95–100	......	......
2-in. screen.........	100	......	......	......	......	......
1½-in. sieve........	......	......	75–90	......	......	95–100
1½-in. screen......	85–100	100	......	......	100	......
1¼-in. screen......	......	90–99	......	......	......	......
1-in. sieve..........	......	......	52–72	40–75	......	not less than 75
1-in. screen.........	50–85	......	......	......	30–70	......
¾-in. sieve.........	......	......	37–57	......	......	not less than 40
¾-in. screen.......	......	......	......	......	......	......
½-in. sieve.........	......	......	17–37	......	......	......
½-in. screen.......	20–45	......	......	......	10–40	......
⅜-in. sieve.........	......	......	10–22	......	......	......
¼-in. screen.......	0–15	2–8	......	......	0–10	......
4-mesh sieve.......	......	......	0–6	0–10	......	0–5
8-mesh sieve.......	......	......	......	0–5	0–5	......

There are also called for in testing under these specifications 28 different sizes of screens and sieves some of which are not manufactured as standard.

The coarse aggregates have their nomenclature somewhat similar to the fine aggregates. Such terms as "roofing gravel," "pea gravel," "bird's-eye," "road gravel," "road surfacing material," "fireproofing aggregate," "paving aggregate," "bridge aggregate," etc., are frequently used in ordering a supposedly more or less definite type of material. Often investigation will disclose a definite specification concealed in the background but in many cases the specification is no more definite than is the nomenclature discussed with reference to fine aggregates.

Indefinitely Worded Clauses—There are certain clauses common to many specifications which are either impossible of literal fulfilment or introduce an indefiniteness most productive of differences of opinion between producer and consumer.

It is, of course, possible to obtain 100 per cent passing any given size by using a screen of that size in production. However, such screens will give a material the major portion of which is considerably below the size specified. Much closer conformity to what is desired will be obtained by allowing from 5 to 10 per cent above the nominal top size with a limit of 100 per cent passing the next larger size if the character of the work requires.

In specifying the bottom limit it is actually impossible to stop 100 per cent on any size in a screening and washing process since a certain proportion of the finer sizes will carry over with the product. This is particularly evident in a separation between coarse and fine aggregates at about ¼-in. since in these fine sizes a certain amount will be carried into the coarse product with the wash water and by adhesion to the larger particles. This will, in general, vary from about 2 to 10 per cent, depending on the character of the material, the method of screening, and the load on the screens.

Such terms as "free from injurious amounts of" or "containing no," applied to silt, clay, organic matter, etc., simply open a fertile field of combat, the violence of which depends upon the personalities and opinions of the parties involved. "Well graded from coarse to fine," while a term that seems difficult to avoid under certain specifications, does introduce the matter of personal interpretation. "To the satisfaction of," or "shall be approved by," when used in an aggregate specification, unless referring to exceptions to the general specification, mean to the producer that he must be well acquainted with the desires and opinions of the "party in power" and that he must take a chance on either a change of authority or the rulings of an assistant changing the aspect of things.

A plant can not be set up that will commercially produce materials meeting this multiplicity of requirements. The great variety of top sizes alone would require a number of separations that would be beyond the possibility of practical operation.

This combination of diversity, indefiniteness and impossibility can have but one result. The producers set up their plants to meet best those specifications which represent the major volume of their demands, and at the same time produce the least possible amount of "waste" or material that will not be absorbed by the market. In case of specifications whose requirements fall outside the possibilities of their plant production, they hit them as near as may be, which in some cases may not be very close.

Although strictly not a matter of specifications, the question of handling, sampling and inspecting aggregates has a vital bearing on both their production and use.

Handling Aggregates—An aggregate as loaded into cars at the plant may meet the mean requirements of a specification and yet by the time this reaches the mixer it may fall outside the limits, due to segregation in handling. This segregation may occur in loading, unloading, stockpiling or distributing. The majority of producers have very considerably reduced the segregation in loading and are constantly experimenting with new ideas in the hope of bettering their results. Segregation in stockpiling may be greatly reduced by using care in building up the stockpile in successive layers, instead of pyramiding it in one operation, while by careful handling during distribution segregation may be held to a minimum.

The seriousness of segregation in stockpiling and handling is clearly shown in the investigation by Walter J. Jerz under the Ray Sand & Gravel Company Fellowship at the University of Michigan in 1925.[1] This consists of a study of segregation of aggregates in loading, stockpiling and handling under normal conditions.

Sampling Aggregates—The taking of samples that are even fairly representative of any given quantity of material is a difficult problem. This is particularly true in the case of coarse aggregates. In loading aggregates the coarser particles in general collect at the sides and bottom, while the fine particles will remain in the center or directly under the point of delivery. This may be materially reduced by the better methods of loading, but can not be eliminated entirely, so that the taking of a sample that is truly representative of a car load means at best a quite appreciable amount of intelligent work on the part of an experienced inspector. Sampling from bins or stockpiles presents similar difficulties of even greater magnitude. When loading belts are used a cross section taken from the belt has been found to give the most accurate sample possible, while samples taken from the end of the loading chutes are almost equally representative. This is brought out at considerable length in the report of Warren E. DeYoung, United Fuel & Supply Company Fellow at the University of Michigan.[2] Mr. DeYoung's report is based on a study to

[1] "The Relation of Handling Operations in the Field to Segregation of Gravel," by Walter J. Jerz. Proceedings of the Twelfth Annual Conference of Highway Engineers, University of Michigan, 1926.

[2] "Investigation to Determine Efficient Methods of Sampling Gravel," by Warren E. DeYoung. Proceedings of the Eleventh Annual Conference on Highway Engineering, University of Michigan, 1925.

determine the most accurate methods feasible in sampling aggregates at the producing plants.

Inspecting Aggregates—Inspection at the point of production has been adopted to a large extent in this district, especially with the more particular consumers. This works out beneficially to both the producer and consumer, as it results in the shipment of material which satisfies specifications and eliminates arguments and rejections at destination. The producer's function is the production of satisfactory material at the plant. What may occur in the line of segregation between plant and mixer is a matter beyond his control but is of vital importance if advantage is to be taken of material which is satisfactory when originally loaded.

In inspection in general there are often practices which are unsatisfactory to both principals. In the past a great deal of material has been accepted or rejected on visual inspection. This may be perfectly satisfactory at the point where the inspector is inspecting several consecutively loaded cars some of which are actually tested and the balance visually inspected by comparison. On the other hand, the visual inspection of a separate quantity of aggregate even by a competently trained observer may be extremely inaccurate. This is particularly true if the material is near the specification limits or if the inspector has been dealing with aggregates under considerably different specifications. This has been very noticeable in the inspection of a coarse aggregate with a 1½-in. top limit by men who have been in the habit of handling material with an upper limit of 2½-in. Cases are not uncommon where material perfectly acceptable under mechanical analysis has been rejected under visual inspection.

Screens and Sieves—Many laboratories engaged in the mechanical analysis of concrete aggregates are apparently unfamiliar with the great variation in results between square sieves and round screens used in testing. The 1928 Report of Committee E-1 of the A.S.T. M.on Methods of Test brings out these variations as shown by various tests. Not only are variations shown from 4 per cent on the ¼-in. to 10 per cent on the 1½-in. openings with the same material, but from 5 to 12 per cent on the same sized opening (1½-in.) with varying materials. In spite of this there have been several cases in this district during the current season where material furnished under a specification written on screens has been tested on sieves, and this is the regular practice in the case of at least one large consumer of aggregates. One of the larger users of aggregates has completely set aside the specification under which bids are received and insists on the producers furnishing material of a totally different mechanical anlaysis.

Conclusions—The designer, specification writer, producer, and ultimate consumer certainly have one common ground—the production of dependable concrete at a minimum cost. This is obvious, as the entire future of the concrete industry depends upon the attainment of this end. The producer can only continue to operate by maintaining

a margin between his cost and selling price. This cost is not only dependent upon his operating cost as such, but upon his waste and rejections under the specifications which he is furnishing.

For over a decade the writer has been engaged in the aggregate industry in Detroit. During the latter part of this period he has been primarily interested in the quality of the material furnished and its adaptability to specification requirements. This has given him a very intimate contact with the complications outlined in this paper. As the production of closely controlled concrete became more general he hoped that the requirements for aggregates would approach at least a partial standardization, but, on the contrary, there has been an ever increasing diversity. Even as this paper is being prepared there are new specifications coming out, covering large quantities of concrete, which are being drafted with apparently no reference to existing specifications nor to the possibilities of the sources of supply.

Concrete of all classes should not require more than two specifications for fine aggregates: one, written as broadly as is consistent with the requirements, for mass work, heavy structures, and the ordinary run of concrete; and one, with possibly closer limits, for work where either particular surface finish or exceptional workability are required. Coarse aggregates of three types, or at the most four, should fill all requirements. One should cover mass work, paving, etc., one for ordinary reinforced construction, and one for fireproofing, or very close reinforcement, with possibly a fourth for work of a very special character.

All of these specifications should be written not only with reference to the technical requirements of the work, but also with due consideration for the practical limitations of commercial production.

Having written consistent and adequate specifications and obtained their fulfilment through plant inspection, steps should be taken to see that the material is handled in such a manner that it still falls within specification limits when it is placed as concrete. This can only be accomplished by competent inspection at all points. It seems obvious that it is a waste of effort to insist on the furnishing of correct material by a producer if it is not handled in such a way as to retain its quality until placed in the work.

On construction requiring very close control it would be distinctly advantageous to ship the coarse aggregate in two or more sizes and combine them in batches at the mixer, thus materially reducing segregation.

Before consistent and dependable results can be obtained there must be a co-ordination of specifications and a closer co-operation between the specification writer and the producer of aggregates. Further, the aggregates, after production, must be correctly handled under competent inspection in order that the material may reach the work in the same condition in which it is originally prepared.

Discussion—Confusion of Specifications for Aggregates

Joseph A. Kitts* (*By Letter*)—The chaotic situation in specifications for aggregates, as described by Mr. Williams, is a general one, not confined to any particular locality, and a source of economic waste and of unfair discrimination. It is a situation detrimental to the concrete industry and Mr. Williams' paper shows the crying need for joint action by the various industries and interests affected. Mr. Kitts.

The adoption of standard screening sizes for concrete aggregates is undoubtedly an economic need of the day and is becoming increasingly important with the increasing use of concrete. I am strongly convinced that the adoption of such a standard would do much to correct the situation as presented by Mr. Williams. Accordingly, it is the purpose of this discussion to offer some suggestions in the consideration of a proper standard of screen sizes.

Round or Square Holes

The round hole is considered the most practical for both the scientific screens of the laboratory and for plant screens, excepting in the small holes of ¼ in. or less. The standard plant screens and the standard laboratory screens should be the same or equivalent.

Round and Square Holes Compared

Round and square holes, of the same average diameter, have the same practical screening effect. If the side of the square is taken as one inch, the average diameter of the opening is $(1+1.4142)/2 = 1.207$ in. and the corresponding round hole should be 1.2 in. in diameter. This relation is shown by the 1928 Report of Committee E-1 of the A.S.T.M., although not observed by the Committee.

Ratio of Sizes

There should be a uniform ratio of successive sizes of screens as, with such a uniform ratio, the fineness modulus, a function of the average diameter of particles, is readily obtained by taking the sum of the proportions retained on such a system of screen sizes. This is of considerable practical and scientific importance in the reproportioning of two, three or several aggregates for a concrete mix.

A ratio of 2 appears to be the most practical one, excepting that in the larger sizes half sizes are often necessary as a measure of economy in using the largest practical maximum size. The ratio between the half and whole sizes would then be the square root of 2, or 1.4142.

* Consulting Concrete Technologist, Kitts & Tuthill, San Francisco.

STANDARD LABORATORY SCREENS

The standard laboratory screens for concrete aggregate are of the square hole type and are given as follows with the equivalent round hole sizes:

	Standard Screen, Side of Square Hole, In.	Equivalent Round Hole Diameter, In.
	3	3.6
	2*	2.4
	1.5	1.8
	1.0*	1.2
	0.75	0.9
	0.375	0.45
No. 4	0.187	Use square
8	0.0937	" "
16	0.0469	" "
30	0.0232	" "
50	0.0117	" "
100	0.0059	" "

* Half sizes.

There is a trend of thought for the round hole screen as the standard in preference to the square hole and there is no important reason why the standard may not be changed both in the shape and size bases.

SUGGESTED SIZES

A uniform system of sizes, in the ratio of 2 for whole sizes and 1.414 between whole and half sizes, corresponding as near as possible to general plant practice, is as follows:

Round Hole Screen Diameter, In.		Equivalent Square Hole Screen, Side of Square, In.	
Whole Size	Half Size	Whole Size	Half Size
12.0	....	10.0	....
....	8.5	....	7.07
6.0	....	5.0	....
....	4.25	....	3.535
3.0	....	2.5	....
....	2.125	....	1.767
1.5	....	1.25	....
....	1.062	....	0.883
0.75	....	0.625	....
....	0.531	....	0.442
0.375	....	0.312	....
....	0.266	....	0.221
Use square			0.156
" "			0.078
" "			0.0395
" "			0.0197
" "			0.0098
" "			0.0049
" "			0.0024

Looking to the eventuality of an international standard, the following sizes are suggested in the metric system:

Round Hole Diameter, Centimeter		Square Hole Side of Square, Centimeter	
Whole Size	Half Size	Whole Size	Half Size
24	....	20	....
....	17	....	14
12	....	10	....
....	8.5	....	7
6	....	5	....
....	4.25	....	3.5
3	....	2.5	....
....	2.125	....	1.75
1.5	....	1.25	....
....	1.062	....	0.875
0.75	....	0.625	....
Etc.		Etc.	

NUMBER OF SIZES FOR CONCRETE

The number of sizes of aggregates used in concrete is increasing from two to four or more and the economical number is about as follows, depending upon the maximum size used:

Maximum Size	Number of Sizes
¼ in.	1 or 2
⅜ in.	2 or 3
¾ in.	3 or 4
1 in.	3 or 4
2½ in.	3 or 4
2 in.	3 or 4
3 in.	4 or 5
4¼ in.	4 or 5
6 in.	4 to 6
8½ in.	4 to 7
12 in.	4 to 8

In the light of the present knowledge of concrete mixtures, 6 to 8 sizes of aggregates would appear to be the economical limit for maximum sizes exceeding 3 in. The maximum size of aggregate that has been handled practically and economically is that of the Exchequer Dam and was 10 in. round hole. As the mixing and placing plant was designed for 7 in. ring maximum, it would appear that 12 in. maximum may be provided for in the future.

UNIFORM GRADING

Where aggregates are separated into several sizes, a tolerance only is needed on the maximum and minimum sizes.

In case separations are not made uniform, grading may be definitely specified by allowing a plus or minus tolerance from the Talbot-Richart grading equation

$$r = 1 - (d/D)^n$$

or the Kitts-Peugh equation

$$r = \frac{1 - (d/D)^m}{1 - (A/D)^m}$$

in which r is the proportion retained on given screen opening of d inches and D and A are maximum and minimum sizes and m and n are coarseness exponents.

It would appear to be a simple matter to arrive at a proper screening standard, and that would do much to correct the present confusion of specifications for aggregates.

Mr. Price. P. W. Price (*By Letter*).—Referring to this excellent paper, I would quote what, in my opinion, is about the most important new suggestion made, although it has been referred to briefly in other papers read at this convention: "On construction requiring very close control it would be distinctly advantageous to ship the coarse aggregate in two or more sizes and combine them in batches at the mixer, thus materially reducing segregation." This, in my opinion, is the next big step in the production of concrete which I think the American Concrete Institute is in a good position to take up and push through to its logical conclusion.

In order to make my point more definite I would express it somewhat as follows:

> On all work where control is undertaken with a view to securing stronger, more durable and more impervious concrete, it is essential that coarse aggregates shall be graded by screening into two or more parts which shall be loaded, transported and delivered to the mixer in separate containers which will insure the proper grading of *every batch* without any segregation except such as might take place within the limits of the screen range specified for each sub-division of the batch.
>
> The proportion of each sub-division of the batch shall be determined by screen analysis of the aggregate as a whole based on any recognized method of securing the greatest density in the resulting concrete. The amount to be drawn from each bin or other container may be measured either by weight or by volume, dependent on the weighing or measuring devices available or as may be specified by the engineer.

As an example of a convenient sub-division of the batch I would suggest the following based on the grading given for "large aggregate" on page 2 of the Specification of the Engineers' Society of Western Pennsylvania, as follows:

Sub-Division A. That passing a 2½-in. and retained on a 1½-in. sieve
Sub-Division B. That passing a 1½-in. and retained on a ¾-in. sieve
Sub-Division C. That passing a ¾-in. and retained on a ⅜-in. sieve

The above is given merely as an illustration of what can be done. It can be done, I hope, without prohibitive cost to the producer. If it adds

somewhat to the cost of concrete I, for one, would think it would be about the best way to spend more money and secure much more in results than by any other single proposal that has been recently made in the concrete industry.

Three main results are obtained:

(*a*) There is no question as to the proper grading of *every yard of concrete placed.*

(*b*) The question of segregation is settled for all time. There need be no more controversies between inspection for grading at the source of supply and inspection for grading at the point of delivery.

(*c*) By taking care of this hitherto weak link in the chain of concrete production and assuming good materials, correct water-cement ratio and reasonable care in placing and curing, the resulting concrete has every chance of being strong, durable and watertight.

A. S. DOUGLASS—I would like to invite Mr. Williams to discuss his own paper by going a little farther and telling what he is trying to accomplish by working with the different customers to get down to a common understanding and a good proportioning basis. **Mr. Douglass.**

L. E. WILLIAMS—I can only say this, briefly: I am not the one doing the work primarily. A committee has been organized in Detroit on which there is representation of the producers, consumers, designing engineers and architects. That committee has been working about five years, possibly six. I feel that they have done a great deal to consolidate specifications, but we are hoping to come out in the not distant future with a set of standard specifications which we feel are adaptable to practically all types of concrete work, with the exception of some very special classes. We are passing that specification around among the different types of men and getting what criticism we can, so that we can come out with something that can be generally adopted. **Mr. Williams.**

Report of Committee E-5 on Aggregates

Two meetings of Committee E-5 on Aggregates were held during the past year, one at Washington, D. C., December 15, 1928, and the other at Detroit, Michigan, at the time of the annual meeting of the Institute.

The principal activity of the Committee has been the further consideration of the proposed Purchase Specification for Aggregates, to which reference was made in last year's report. It has been the special object of the Committee to harmonize this specification in so far as possible with the corresponding specification of the American Society for Testing Materials.

The specification as amended is again presented to the Institute with the recommendation that it be adopted as a tentative standard.

The Committee recognizes that under certain conditions the requirements covering concrete strength as given in Paragraphs 6 and 8*b* may not prove entirely adequate. This is true in cases where both fine and coarse aggregate are of doubtful quality and the final acceptance of each depends upon compliance with the corresponding concrete strength requirement. The Committee is considering this matter carefully and, pending further revision, recommends that the concrete strength clause as now worded remain in the specification.

Committee E-5 also gave consideration to the matter of standardizing methods of testing. It is the sense of the Committee that, where suitable published standard or tentative standard methods of testing of the American Society for Testing Materials or other organizations are in existence, the American Concrete Institute should not, in general, publish such methods in full but should refer to them in its specifications by title and suitable reference number only. The Committee accordingly withdraws the proposed tentative method of test for abrasion of gravel published in last year's report, this method being identical with the corresponding A.S.T.M. method.

During the coming year, the Committee will continue its work in connection with the purchase specification, realizing that, although in its present form it represents probably the latest engineering thought upon the subject, there are still many doubtful points, particularly as regards suitable test limits for aggregates for various uses, which should be investigated and cleared up as soon as possible.

R. W. Crum, *Chairman.*
F. H. Jackson, *Secretary.*

Proposed Purchase Specification for Concrete Aggregates

Submitted by Committee E-5, Aggregates

(The A.C.I. Tentative Purchase Specifications for Concrete Aggregates, E-5 A-26 T, as revised at a meeting of Committee E-5, December 15, 1928, and adopted by the convention as tentative—E-5 A-29 T.)

Scope of Specification

The purpose of this specification is to provide a standard form for use in writing specifications to govern the purchase of concrete aggregates. It is presumed that the individual user will insert test limits that will meet his conditions as to projected use and characteristics of available materials.

The recommendations for test limits included in the specification are intended to govern the quality and size of aggregates proposed for use in Portland cement concrete for general purposes, and are based primarily upon considerations of strength. Where it is essential that the concrete possess additional characteristics, such as resistance to fire, action of sea water, unusual climatic conditions, etc., such additional requirements for the aggregates as may be found necessary should be inserted in the specifications.[1]

In general, the test limits included show the ranges within which use is recommended. The individual user, in preparing his specifications, should include specific limits which best meet his local conditions, within the ranges herein recommended.

Quality of Aggregates

It is recognized that for certain purposes satisfactory results may be obtained with materials not conforming to these specifications. In such cases, the use of fine and coarse aggregates not conforming to these specifications should be authorized only under special provisions based upon laboratory studies of the possibility of designing a mixture of materials to be used on the job that will yield concrete equivalent to the specified mixture made with material complying with these specifications in all respects.

Fine Aggregate

General Characteristics.

(1) Fine aggregate shall consist of sand or other approved inert materials with similar characteristics, or a combination thereof, having hard, strong, durable particles, and shall conform to the requirements of these specifications.

[1] The Committee is at work on this problem and will furnish suggested test requirements covering the physical characteristics of aggregates for special purposes as addenda to these specifications as soon as possible.

Deleterious Substances.

(2) (*a*) The maximum percentages of deleterious substances shall not exceed the following values:

	Per Cent by Weight
Removed by decantation	3
Shale	1
Coal	1
Clay lumps	1
Other local deleterious substances (such as alkali, mica, coated grains, soft and flaky particles)	..

NOTE.—It is recognized that under certain conditions maximum percentages of deleterious substances less than those shown in the table should be specified.

(*b*) The sum of the percentages of shale, coal, clay lumps, soft fragments and other deleterious substances shall not exceed 5 per cent by weight.

(*c*) All fine aggregate shall be free from injurious amounts of organic impurities. Aggregates subjected to the colorimetric test for organic impurities and producing a color darker than the standard shall be rejected unless they pass the mortar strength test as specified in Section 4.

Grading.

(3) (*a*) Fine aggregate shall be well gradea. rom coarse to fine and when tested by means of laboratory sieves shall conform to the following requirements:

	Per Cent
Passing a ⅜-in. sieve	100
Passing a No. 4 sieve	(85) to (100)
Passing a No. 16 sieve	(45) to (80)
Passing a No. 50 sieve	(2) to (30)
Passing a No. 100 sieve	(0) to (5)

NOTE.—Figures in parentheses are suggested as limiting percentages but they may be altered within these limits to suit local conditions.

Mortar Strength.

(4) Fine aggregates, when subjected to the mortar strength test, shall have a tensile or compressive strength at the age of 7 and 28 days, equal to or greater than that developed by mortar of the same proportions and consistency made of the same cement and standard Ottawa sand.

Concrete Strength.

(5) Fine aggregates passing the maximum size sieve requirement, but otherwise failing to meet the requirements herein provided for grading or mortar strength, may be used if, when tested in combination with the coarse aggregate to be used in the work, in the proportion specified for the class of concrete under construction, the crushing or transverse strength of the concrete at the end of 7 and 28 days is at least equal to that of concrete of the same proportions and consistency made

with the same cement and coarse aggregate in combination with a fine aggregate meeting all the requirements of these specifications.

COARSE AGGREGATE

General Characteristics.

(6) Coarse aggregate shall consist of crushed stone, gravel, blast-furnace slag, or other approved inert materials of similar characteristics, or combinations thereof, having hard, strong, durable pieces, free from adherent coatings and conforming to the requirements of these specifications.

Deleterious Substances.

(7) (*a*) The maximum percentages of deleterious substances shall not exceed the following values:

	PER CENT BY WEIGHT
Removed by decantation	1
Shales	1
Coal	1
Clay lumps	¼
Soft fragments	5
Other local deleterious substances (such as alkali, friable, thin, elongated or laminated pieces)	..

NOTE.—It is recognized that under certain conditions maximum percentages of deleterious substances less than those shown in the table should be specified.

(*b*) The sum of the percentages of shale, coal, clay lumps and soft fragments shall not exceed 5 per cent by weight.

Grading.

8. (*a*) Coarse aggregate shall be well graded, between the limits specified, and shall conform to the following requirements:

PASSING			PER CENT BY WEIGHT
—— inch sieve	(maximum size)	not less than	(95)
—— inch sieve	(intermediate size)	not less than	(40)
	(½ maximum size)	not more than	(75)
—— inch sieve	(intermediate sizes)	not less than	..
	(as needed)	not more than	..
No. 4 sieve		not more than	0 to 10

NOTE.—Where a range is shown, the engineer should use an appropriate figure within the limits recommended. The figures in parentheses are recommended but may need to be altered to suit local conditions.

Concrete Strength.

(*b*) Coarse aggregate failing to meet the grading requirement may be used, if, when tested in combination with the fine aggregate to be used in the work, in the specified proportions, the crushing or transverse strength of the concrete at the end of 7 and 28 days is at least equal to that of concrete of the same proportions and consistency made with the same cement and fine aggregate in combination with a coarse aggregate meeting all the requirements of these specifications.

Weight of Slag.

9. Blast furnace slag that meets the grading requirements of these specifications shall conform to the following minimum weight requirements:

General concrete	65 lb. per cu. ft.
Concrete subject to abrasion	70 lb. per cu. ft.

Durability.

10. Coarse aggregate shall pass a sodium sulfate accelerated soundness test, except that aggregates failing in the accelerated soundness test may be used if they pass a satisfactory freezing and thawing test.

NOTE.—Many engineers feel that an abrasion test for coarse aggregate to be used in concrete subject to abrasion is important, but no test limits are specified, due to the status of knowledge concerning suitable specification limits for this test.

METHODS OF SAMPLING AND TESTING

Methods of Testing.

11. The properties enumerated in these specifications shall be determined in accordance with the following methods of test of the American Society for Testing Materials, except as specified in Paragraphs (*e*), (*g*), (*i*) and (*j*).

(*a*) *Sampling*—Standard Methods of Sampling Stone, Slag, Gravel, Sand and Stone Block for Use as Highway Materials, Including Some Material Survey Methods (Serial Designation: D 75).

(*b*) *Sieve Analysis*—Standard Method of Test for Sieve Analysis of Aggregates for Concrete (Serial Designation: C 41).

(*c*) *Decantation Test*—Tentative Method of Decantation Test for Sand and Other Fine Aggregates (Serial Designation: D 136–22 T).

(*d*) *Organic Impurities*—Standard Method of Test for Organic Impurities in Sands for Concrete (Serial Designation: C 40).

(*e*) *Mortar Strength*—Methods of Making Compression and Tension Tests of Fine Aggregate for Concrete as Adopted by the American Association of State Highway Officials, and described in U. S. Department of Agriculture Bulletin 1216, Revised.

(*f*) *Compressive Strength*—Standard Methods of Making Compression Tests of Concrete (Serial Designation: C 39).

(*g*) *Soundness*—Method of Test for Soundness of Coarse Aggregate (Sodium Sulfate Soundness Test) as adopted by the American Association of State Highway Officials and described in U. S. Department of Agriculture Bulletin 1216, Revised.

(*h*) *Freezing and Thawing*—Method of Freezing and Thawing Tests of Drain Tile as described in the Standard Specifications for Drain Tile (Serial Designation: C 4).

(*i*) *Shale and Coal*—Method of Test for Percentage of Shale in Aggregate as adopted by the American Association of State Highway Officials and described in U. S. Department of Agriculture Bulletin 1216, Revised.

(*j*) *Soft Fragments*—Method of Test for Quantity of Soft Pebbles in Gravel as adopted by the American Association of State Highway Officials and described in U. S. Department of Agriculture Bulletin 1216, Revised.

(*k*) *Weight of Slag*—Standard Method of Test for Unit Weight of Aggregate for Concrete (Serial Designation: C 29).

(*l*) *Abrasion*—If abrasion tests are made the following methods of tests are recommended:

(1) *Abrasion of Gravel*—Tentative Method of Test for Abrasion of Gravel (Serial Designation: D 28 T).

(2) *Abrasion of Rock or Crushed Slag*—Standard Method of Test for Abrasion of Rock (Serial Designation: D 2), except that, for specific gravities lower than 2.2, a 4,000-g. sample shall be used.

This report has been submitted to letter ballot of the committee, which consists of 15 members, of whom 13 have voted affirmatively, none negatively, and 2 have refrained from voting.

F. H. JACKSON, *Secretary*,
Committee E-5 on Aggregates.

Mr. Powers.

T. C. Powers* (*By Letter*).—The writer objects to the fine aggregate specification, sections (2c) and (4), on the grounds that (1) it often causes the rejection of good material, (2) it might permit the use of unfit material, and, (3) the mortar test as described does not determine the suitability of the sand either from the standpoint of physical quality or suitability of grading. These objections will be discussed in the order given.

Section (3a) recognizes the fact that sands covering a considerable range of grading may be used successfully in concrete mixtures. The specification covers fine aggregate graded from zero to ⅜, fineness modulus about 3.80 as a maximum, to one graded zero to No. 4, fineness modulus about 2.45, as a minimum. As stated in Section (4), sands within this range are to be compared with standard Ottawa sand, by tests on mortars identical as to mix and consistency. Presumably, a sand which develops as much strength as the standard mortar is of suitable quality. If the strength is lower, the particles are apparently unsound, or carry organic matter, or are in some other way unfit for use in concrete. As a matter of fact, the sand may be of the best physical quality, and may not carry any injurious matter at all, and still fail in the test. It is obvious that if we wish to determine the effect of one variable, all other variables having the same effect must be under control. Among the several variables which effect the potential strength of concretes and mortars, the water-cement ratio is the most important. With any given mix and consistency, the water-cement ratio is a function of the grading of the aggregate. The finer the aggregate the higher the water-cement ratio and the lower the strength. Taking the specification as it stands we find the following situation for any given mix used in the test:

Variables	"Unknown" Sand	Standard Sand
1. Grading	Fineness modulus may vary from 2.45 to 3.80	No. 30 to No. 20
2. Water-cement ratio (w/c) (More accurately, voids-cement ratio)	Function of fineness modulus	Constant
3. Quality	Unknown	O.K.
4. Strength	Function of w/c and of quality	Constant

From this it appears obvious that strength is a function of both quality

* Bureau of Water Works, Portland, Oregon.

and water-cement ratio, and that no method of telling the effect of the variables separately is apparent. For example, in a 1:5 mix, sand graded O–4, fineness modulus 2.45, the water cement ratio was 1.42. *The same material* recombined so as to have a fineness modulus of 3.80 decreased the water requirements to a water-cement ratio of 1.22. In the first case the strength was 1200 lb. per sq. in., and in the second case, the strength was 1600 lb. per sq. in.*. If the strength obtained from the coarse grading were just equivalent to the standard, then the finer grading would be rejected on the basis of a 0.75 strength ratio, and yet the material is identical except as to grading.

Again, tabulation of hundreds of tests made in Oregon using the 1:3 mortar test, showed that the average sand, graded zero to No. 4, would not pass the test if the fineness modulus was below 3.00 due to the high water-cement ratio necessary to produce the standard consistency. Many sands with a fineness modulus below 3.00 passed the test, but usually they were not in the zero-to-four classification, that is they were *small* sands but not necessarily finely graded.

This means, then, that the fine sands are handicapped in the test, and that the coarse sands are apt to pass. In Oregon, it means that practically all sands from 2.45 to 3.00 would not pass the test. Yet, in lean mixes where the water-cement ratio is greater than about 1.00, most sands coarser than 3.00 will not make satisfactory concrete without the use of admixture. Experience and theory agree that the leaner the mix, the finer the sand should be, and vice versa. That is, for any given mix and maximum size there is a certain maximum fineness modulus of the combined aggregate which will produce a workable mixture. Likewise, there is a certain maximum fineness modulus for the sand in the mix. This point was brought out in Abrams' Bulletin No. 1, and experience has corroborated it. Increasing the proportion of a sand too coarse for the mix will not compensate for lack of fines.

Summed up, the first objection is that the mortar test does not eliminate the effect of variation of water-cement ratio due to variation in grading of the sand, and that with quality constant it automatically rejects fine sands and passes coarse ones, while fine sands are often the best for the purpose.

The second objection is that it is possible for an unfit material to pass the test. Suppose that a sand graded (O–4), fineness modulus 3.80, failed in the colorimetric test. It is then subjected to the mortar test and passes with, say, 105 per cent of standard strength. It could then be accepted for use, according to the specification. Let us assume that, in reality, the sand contained enough organic matter to cause a 20 per cent reduction in strength below normal for the water-cement ratio which the grading of sand and mix required. This would indicate that the test should have 125 per cent of the standard strength, but the test does not show this, because no attention is given the water-cement ratio. To further illustrate:

* Data from Bulletin 137, University of Illinois, by Talbot & Richart, Table 2 and Fig. 44. The water ratios are voids-cement ratios in Abrams' units.

	STANDARD SAND	UNKNOWN SAND
Mix	1:3	1:3
w/c	0.70	0.6
Strength	3500 lb.	3650
Consistency	Normal	Normal

The unknown sand having a water ratio of 0.60 had only to develop as much strength as the standard mortar having a water ratio of 0.70. The unknown sand had normal consistency at a lower water-cement ratio because of its grading.

The important point is that such a sand is able to pass the test because of the grading and in spite of its injurious organic content.

Having accepted the sand for use, we combine it with coarse aggregate in such proportions as to produce a certain fineness modulus of the combined aggregate, (Abrams' Method), or we combine it according to some arbitrary mix. Let us consider the first alternative:

By the fineness modulus method the sand is combined entirely on a basis of its grading, taking into consideration the grading of the fine aggregate, of the coarse aggregate, and of the combined aggregate to be produced.

Once the combination is made we are no longer dealing with a sand graded (O–4), fineness modulus of 3.80, but with an aggregate graded from, say, (O–½) having a fineness modulus of 5.80. The sand is now an integral part of the combined aggregate. The water requirement of the batch is determined by the grading of the combined aggregate, not by the sand alone. Thus, the advantage in grading which enabled the sand to pass the test is lost when it becomes but a part of the total aggregate. If we expect a strength of 2000 lb. per sq. in. when $w/c = 1.00$, the sand as described might reduce the strength to 1600 lb., due to the organic matter it contained.

If we adopt the second alternative, and use the sand in an arbitrary mix such as 1:2:4, it is possible that not only will the injurious organic matter lower the strength, but also that *the very grading which enabled the sand to pass the test may be a disadvantage in an arbitrary mix.* For example:

Given:

Coarse Aggregate	
Fineness modulus	7.50
Maximum size	2 in.
Fine Aggregate	
Fineness modulus	3.80
Maximum size	⅜ in.
Mix	1:2:4

The above materials combined in a 1:2:4 mix produces a combined aggregate having a fineness modulus of 6.28. It will be found that this combination produces a harsh mix and probably would not develop maximum strength. Besides, there is the organic matter to lower the strength. A good sand having a fineness modulus of about 2.80 would probably fail

the mortar test, but would be the coarsest sand which would produce satisfactory concrete with this particular combination of materials.

The second objection, summed up, is that a poor sand might pass the test if it is coarse, but its coarseness may even be a disadvantage in concrete mixtures, depending on the richness of the mix. The poor quality would be detected in concrete because the arbitrary criterion used in the test is not used for concrete.

What, then, does the test tell us? The writer believes that, as it is now interpreted, it gives no useful information, and is even deceptive.

This subject has been given considerable study and experiment, but as yet we have been unable to evolve a satisfactory test to supplant the old one. The next best thing seems to be to change the interpretation of the present test. An outline of the plan is presented in the following:

The water-cement ratio strength relation is of the form

$$S = \frac{A}{B^{w/c}}$$

Where, for given conditions of tests S equals compressive strength; A and B equal constants depending on quality of materials and on other minor factors and w/c = water-cement ratio.

By testing three or more mixes 1:1; 1:2; 1:3, etc., of cement and standard Ottawa sand, the values of the constants in the above equation could be determined and used as the standard. The "unknown" sand would then be tested in the same series of mixes, and the quality compared by comparing the positions of the water-cement ratio curves, or by comparing the values of the constants in the equation. Mixtures should be of thoroughly plastic consistency to minimize air voids, or, if preferred, the voids-cement ratio may be used instead of the water-cement ratio.

If the same value of constant A be used for both the standard and "unknown" equation, the quality of the sand could be expressed as the ratio B/B' where B is the constant for the standard equation and B' is the constant for the "unknown" equation.

In tests of this nature, refined methods must be used in determining the water-cement ratios. To do this accurately involves careful measurements. Not only must water losses during the molding process be taken into account but also water losses subsequent to molding due to settling of the solids, leaving free water above. This effect is most noticeable in mixtures made with clean, coarse sand such as standard Ottawa.

The best method we have found for accounting for water losses is as follows: The original water-cement ratio as mixed is carefully determined, making allowance for absorption. The mixture is then placed in the water-tight molds until the mold is level full, being careful not to strike off water in finishing the top. Since any effective change in water-cement ratio after molding involves the reduction of volume of concrete by the amount of water lost, the water loss may be determined by measuring the volume loss due to settling of solids. Settlement, if any, is practically complete within 3 hours after molding. About 6 hours after molding, the

water, which was left above as the solids settled, will be absorbed into the concrete, due to absorption by cement during the hardening process. The water loss can not be determined by loss of weight, for this reason.

After the water is absorbed, the molds may be refilled with water, measuring the amount by means of a burette or pipette. A colored liquid enables more accurate work because of its visibility. Knowing the original volume, and percentage composition by absolute volume of the material in each mold, and the loss of water by the above method, the actual water-cement ratio can be calculated.

While this method would probably result in a fair comparison with the standard mortars, it is still questionable as to whether a sand such as standard Ottawa sand should be used as the criterion. Due to its improper grading, if the mix is made leaner than about 1:2, the mortar does not develop normal strength. At equivalent water-cement ratios, most natural sands will develop greater strengths than standard Ottawa at mixes 1:3 or leaner. If the mixes be kept richer than 1:3, the strengths at 28 days may be as high as 6000 lb. per sq. in. Is it necessary for a sand intended to be used in 1500 or 2000 lb. concrete to stand a 6000 lb. test? Or, should the standard be based on the work which the sand is expected to perform?

Mr. Meacham. James A. Meacham* (*By Letter*)—Relating to the proposed purchase specification for concrete aggregates submitted by Committee E-5: paragraph 2*a* of the specification for fine aggregates includes "coated grains" in the group of local deleterious substances for which it is assumed the engineer will assign a maximum permissible percentage.

It has been our experience that the condition of coated grains is ordinarily a matter which affects all of the grains of the sand and not any particular percentage. The degree of coating, of course, will vary with different materials and it is the degree of coating as applied to all the grains, rather than the percentage of grains which are coated, which will adversely affect the concrete made from the sand in question.

Paragraph 2*b* establishes a maximum total percentage of objectionable material. The use of the inclusive phrase "and other deleterious substances" prompts us to inquire if material "removed by decantation" is to be included in this total. It is, of course, omitted from the list as drawn and the inference from the similar provision in the case of coarse aggregate indicates that the omission is intentional. The point, however, is not clear.

We note that the grading of fine aggregate recommended by the committee (paragraph 3*a*) does not set up a value for the No. 8 sieve. In the specifications offered by the Portland Cement Association for the guidance of those interested in using the water-ratio theory of concrete design and control, the only size considered of sufficient importance to receive a definitely specified limit is the material retained on the No. 8 sieve. It has been our experience that the material retained on the No. 8

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* Specification writer, H. K. Ferguson Company, Cleveland, Ohio.

(in conjunction with the fineness modulus) is an excellent criterion by which to judge the aggregate from an economical standpoint, bearing in mind that a fineness modulus for the whole aggregate ranging between 4.75 and 5.50 will ordinarily produce concrete of the highest economy consistent with workability.

The proposed specification does not set up a requirement for fineness modulus, either for fine or coarse aggregate. Based upon the work of those responsible for the development of the water-ratio theory of concrete control, the fineness modulus of an aggregate is an important consideration in establishing the suitability of that aggregate for concrete work.

We note (paragraph 4), that fine aggregate is required to equal the strength of standard Ottawa sand when subjected to the mortar strength test. We have not considered this a sufficiently high strength requirement in view of the fact that the very uniformity which makes Ottawa sand an excellent basis for comparison adversely affects its strength to a degree which makes it unsuitable as a standard of strength.

In the case of the specification for coarse aggregate (paragraph 7*a*), "thin and elongated pieces" are included in a final group of local deleterious substances, the maximum percentage of which is to be established by the engineer. We would like to suggest that, in our experience, practically every gravel aggregate, if inspected carefully, will show from 5 to 10 per cent of pieces which may be termed thin or elongated. The presence of thin or elongated pieces is of course objectionable if in excess, particularly because of increased cost of placing the resulting concrete. However, if the allowable percentage of this group of local deleterious susbtances is set at a figure which would recognize the normal content of thin or elongated pieces, the value of the specification is impaired, since aggregates decidedly objectionable from the standpoint of alkali, friable or laminated material, would be acceptable under the specification, provided the content of thin elongated pieces were below normal.

The proposed specification (paragraphs 5 and 8*b*), with certain exceptions, permits the use of any aggregate which will produce a concrete of a strength equivalent to that which may be secured from another aggregate which does fulfill the requirements of the specifications. Without doubt, the most substantial criterion for comparing concrete aggregates is the quality of the concrete which they produce. The multiplicity of aggregates available for use has made it practically imperative that specifications be drawn with a degree of flexibility greater than would otherwise be desirable. As a result, it is neither beyond reason nor a reflection upon the work of the committee to assume that an aggregate might be found or produced which would so take advantage of the several limits allowed by the specification as to produce a concrete below expected strength.

In view of the above we particularly commend the use of a strength requirement for the concrete which an aggregate will produce. We suggest, however, that the requirement for strength should be specified as a definite value reflecting the particular needs of the work at hand rather

than based upon the rather uncertain strength which a supposedly proper aggregate is expected to produce.

Mr. Jackson. F. H. Jackson (*during his presentation of committee report*)—T. C. Powers, of the Bureau of Water Works of Portland, Ore., takes exception to paragraphs 2*c* and 4. Paragraph 2*c* is the one in which the cleanliness, or at least the freedom from organic matter in fine aggregates, is determined by means of the color test, subject to a strength-ratio test as a final check. Paragraph 4 is the time-honored mortar strength-ratio test for concrete sand. I may say that the committee is in full agreement with Mr. Powers as to his criticism of the mortar strength test. We have recognized and every one recognizes that this is not a satisfactory test, because, although it is designed to determine the quality of sand grains in fine aggregates, it is so dependent upon the grading that there are two variables constantly entering, and it is frequently impossible to tell which is causing any reduction which may be noted in strength. However, as Mr. Powers has stated in his discussion and as we recognize, we have not been able to dispense with it simply because we have never found a substitute. Therefore, this particular clause appears in the purchase specifications with the idea that it will be supplanted as soon as we can work out a substitute method of test to determine the quality of sand grains. A number of such substitutes are being worked on now, most of which utilize the water-cement strength ratio principle.

I wish also to call attention to the so-called concrete strength clauses, 5 and 8*b*—one governing fine aggregates, and the other governing coarse aggregates. The intent there is to provide a means of final determination as to suitability of an aggregate which may not comply with the grading requirements set up in the specifications and also in the case of fine aggregates which may not comply with the mortar strength requirements, when an arbitrary proportion specification has been set up. In other words, there is no question of defining a mix or changing the proportion, but the final determination by comparing the concrete strength of the fine aggregate under question with the concrete strength of another fine aggregate which meets all the requirements of the specification. The committee recognizes that these two paragraphs are defective in one respect, in that they do not take care of the situation where both the fine and coarse aggregates are under question. Both materials may be defective for any given job, and we compare one with the other and then back again and do not have a final answer. I presume, in the majority of cases, a final comparison would be made against a concrete made up with fine and coarse aggregates, both of which comply with the provisions of the specification, though it is not so stated in this draft. I desire to call attention to the fact that the committee recognizes this defect in the specifications. We know it is not perfect and we are trying to make it better every year. We have worked on it four or five years and intend to continue the work until we have the best purchase specifications for aggregates which it is possible for our committee to get out.

Continuity as a Factor in Reinforced Concrete Design

Hardy Cross*

Introduction

The purpose of this paper is to explain a rapid and accurate approximate method for analyzing continuous girders and frames of reinforced concrete for bending moments, shears, and reactions and to bring together certain questions and considerations which bear on the interpretation of such analyses. The whole subject has often been confused on the one hand by too crude approximations and on the other by too fussy mathematical theory. The consequence of the mathematics has frequently been that the theory has occupied so prominent a position as to shut out from consideration certain practical aspects, and the problems solved often bear little resemblance to actual construction.

It is important to recognize that continuity in reinforced concrete construction is a fact—not a theory. It is obvious to anyone that a concrete beam can not bend without deforming the girders and columns connected to it. It is more important to recognize this fact clearly than it is to evaluate the effect with great precision. Provision must be made in some way for this bending and the resulting shears.

Perhaps the title chosen is too broad, for the paper raises rather than discusses some of the questions involved, and it has been found necessary to omit several phases of the subject entirely, especially the effect of specifications on economical type of design.

The Conventional Rules

Scarcely any idea seems more definitely entrenched in concrete literature and practice than the conventional moment coefficients. Until recently these have been confined in most cases to $\frac{1}{10}$ in the end span and $\frac{1}{12}$ in interior spans. Later specifications suggest $\frac{1}{16}$ at the center of the beam in special cases. The writer has never seen a complete history of the evolution of these coefficients. They probably originated from the work of Winkler about 1885 and have been inherited by English and American engineers from that source.

These rules are generalizations from elastic analysis of a series of beams of equal spans with ratios of live to dead load varying between 1 to 3 and 3 to 1. They should not be extended beyond this. An important point is that they depend upon elastic analysis and do not

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carry weight of independent authority or experimental evidence. They have recently been attacked in a number of technical articles and have been criticized in some recent texts. Careful examination of them shows that they are fairly satisfactory for such regular cases as usually occur in building practice. They should not be extended to irregular span lengths, they should not be extended beyond the range of live-dead ratios indicated above; within these ranges they are not so bad.

It seems unfortunate that the practice of designing live and dead load together became established in concrete practice but, having become established, it gives a reasonably satisfactory basis for comparative designs in certain standard cases.

There are many cases in which the designer will not be satisfied with these coefficients. It is becoming more common to make complete studies in cases of irregularity. The method now to be suggested is approximate but will give needed results within a few per cent of the "exact" results.

Note that the normal problem in design is the determination of maximum moments and shears and sometimes of maximum reactions rather than the determination of the moments or shears for any single condition of loading.

MAXIMUM MOMENTS IN CONTINUOUS SPANS

In securing data to draw curves of maximum moments we load one span at a time successively with live load and determine the end moments. In order to do this we find first the end moments in the span on the assumption that the ends are fixed.

Fixed end moments may be found in beams of uniform section from the general rule that the moment at one end of a fixed ended span due to a concentrated load is equal to the moment which would exist under that load if the beam were simply supported, times the proportional distance of the load from the other end of the span. If several concentrated loads occur the end moment should be determined for each and the results added. For uniformly distributed loads the fixed end moments are $\frac{1}{12}$ WL. For cases of non-uniform distributed load either simple calculus may be applied or the load may be broken up into a series of concentrated loads and the end moments computed as just indicated. The effect of haunches is discussed later.

In general these fixed end moments will not be balanced on the two sides of a joint. Distribute the unbalanced moment at the joint among the connecting members in proportion to the ratio of moment of inertia to span length. Carry over with opposite sign to the next joint one-half of such distributed moment and there distribute again in proportion to the I/L values and carry this out to the end of the series. Add the distributed moments to the original fixed end moments, and this will give the moments at the joints.

This procedure should be followed for live load in each separate span. Addition of the results will give the effect of live load in all spans and if

the distribution of the dead load is the same as for the live, dead-load moments may now be found by proportion. Otherwise they must be computed separately.

The whole procedure may be checked by separately distributing unbalanced moments due to full uniform load.

In drawing curves of maximum moment, draw one curve of moments for dead load in all spans plus live load in alternate spans beginning with the first and another curve of moments for dead load in all spans plus live load in alternate spans beginning with the second. The portions of these two curves which lie between points removed about 0.2[1] of the span

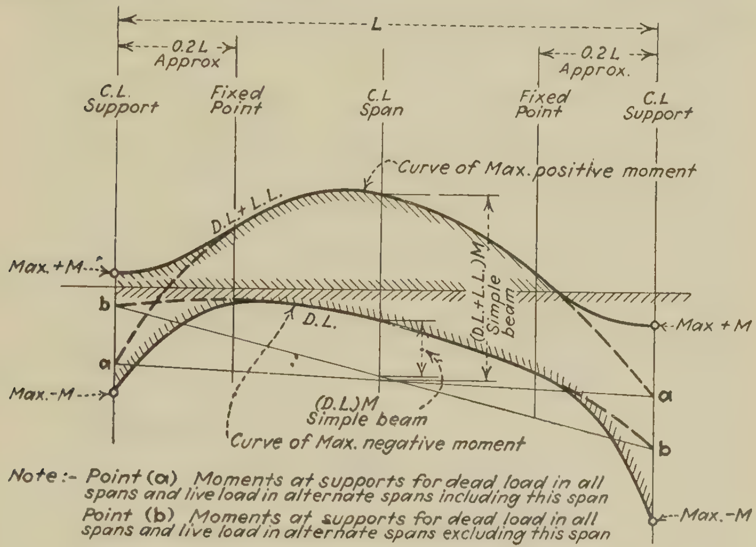

FIG. 1. A PROCEDURE FOR CONSTRUCTING CURVES OF MAXIMA—SINGLE SPAN

length from the supports are portions of the curves of maximum positive and maximum negative moments.

At the supports maximum positive and maximum negative moments may be found by adding all positive or all negative live load moments just found at these supports to the dead load moment. Curves of maxima may now be completed by connecting the curves just mentioned to the maxima at the supports.

The whole construction is shown in Fig. 1 for a single span and in Fig. 2 for a series of spans. The shape of the curves will depend on the type of loading, the curves of moment for simple beams being drawn on the *a-a* or *b-b* lines (see Fig. 1) as bases. Both Figs. 1 and 2 are drawn for uniform loads. If the loads are concentrated at panel points, the

[1] These points are the fixed points used by Fidler, Ostenfeld, and others.

curves of maximum will break once at each panel point, once at each fixed point and once between each fixed point and the support for each panel point (see Fig. 5).

Signs.—The usual conventions of signs should be used in the girders.[1] The writer finds it convenient to apply this convention of signs also to the columns. The girders are read from the bottom up, the columns from the right hand side of the sheet—in other words, girders and columns are looked at as a drawing is ordinarily read. Moments in the girders are written parallel to the girders; moments in the columns parallel to the columns, above the column at its top and below at its bottom.

A joint will be balanced when the total moment on each side (left and right sides looking from the bottom of the page) of it is the same both numerically and in sign.

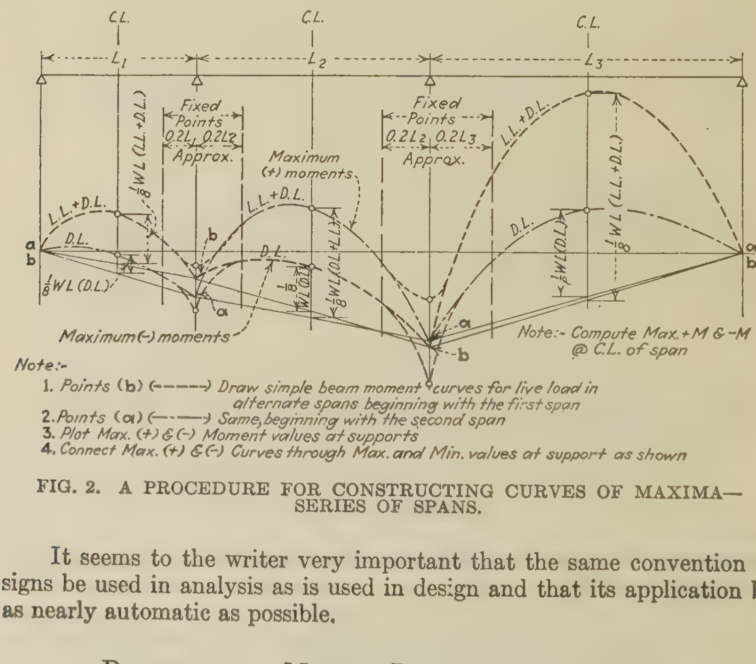

FIG. 2. A PROCEDURE FOR CONSTRUCTING CURVES OF MAXIMA—SERIES OF SPANS.

It seems to the writer very important that the same convention of signs be used in analysis as is used in design and that its application be as nearly automatic as possible.

DERIVATION OF MOMENT DISTRIBUTION PROCEDURE

The writer has used the method of moment distribution in various exact and approximate forms and has explained different phases and applications of it for the past five years.[2] Of all the variations that he has used, the above procedure seems the simplest and most rapid adequate method for concrete structures.

[1] Positive moment "sags" the beam; negative moment "hogs" the beam.

[2] Various aspects of moment distribution are discussed in the writer's Notes on Statically Indeterminate Structures, mimeographed, 1926.

It is evident that fixed end moments would exist if all joints were held so that they could not rotate and that the joints tend to rotate because the fixed end moments are in general not balanced.

Consider the influence of any one of these unbalanced moments on the system. If all of the members connected to that joint were fixed against rotation at their far ends it is easily shown from the theory of elasticity that this unbalanced moment would be distributed among them in proportion to their I/L[1] values and also that there would be produced at the other end of each member a moment of opposite sign and equal to one-half of this distributed moment.[2] If any one of these connected members were free at its far end, its stiffness (measured by the moment necessary to produce unit rotation) would be only three-fourths as great as if it were fixed ended.[3] This would alter somewhat the distribution of the unbalanced moment among the connected members. Some of the members may be fixed, some may be free and some may be continuous at their ends and the exact distribution of these unbalanced moments among them will depend upon this degree of fixation.

We will assume all members at any joint to be partly fixed at their other ends—to use a loose term, half fixed and half free. If they are completely fixed, they will be about 14 per cent stiffer and if actually free they will be about 14 per cent less stiff. But in reinforced-concrete construction the relative values of EI are uncertain to a greater extent than 14 per cent and the fixed end moments are themselves uncertain to the extent of 5 or 10 per cent. The quest for precision greater than is given by this method seems to neglect essential physical elements in the problem.

The approximate method indicated above in which the unbalanced moments at the joints are distributed among the connecting members in the ratio of their I/L values irrespective of the stiffness of the far ends gives results within the limits of accuracy possible from the physical data—about 5 per cent of the unbalanced moment at the joint.

It is possible that some engineers will not agree that this is satisfactory. Those who wish greater "precision" may vary the increase or decrease according to the stiffness of the connecting spans. In the next section an exact method of doing this is indicated.

The writer feels satisfied that any such attempt at precision in concrete structures is entirely unwarranted. In many cases, after the fixed end moments are computed, it is scarcely worth while to pick up a slide rule to get such facts as a designer needs.

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[1] All fixed ended beams loaded with moments at their ends will have moment curves of the same proportions. The end slope is proportional to the end shear due to the M/I curve as a load, if the beam formula holds. Hence the end slope varies as ML/I and the moment necessary to produce a given end slope varies as I/L.

[2] Since the end slope is proportional to the end shear due to the M/I curve as a load, the end slope of a simple beam AB with a moment at one end A, is twice as great at the end A as at the end B. Hence a moment at B half as great as that at A and of opposite sign is needed to eliminate slope at B.

[3] This follows from the concept of moment distribution. Thus rotate end A of a beam, end B being fixed. Hold A and release B and carry over half of M_b with opposite sign. At A we now have $M_a - \frac{1}{2}M_b = M_a - \frac{1}{2}\frac{M_a}{2} = \frac{3}{4}M_a$.

In general the absolute error in the distributed moment in per cent of the unablanced end moment will be less than one-fourth of the percentage error in relative stiffness. In Fig. 3 is shown the error in per cent of the unbalanced moment in the extreme case of one girder free and one fixed for various ratios of I/L of the two spans.

This fact, that a considerable error in the relative stiffness of the members makes so small a difference in the result is to the writer the most interesting and important fact connected with moment distribution. If this were not true, moments due to continuity in concrete would, because of the large and erratic variations in E, be as indeterminable as some theorists have alleged them to be.

Comparison of Exact and Approximate Values.—In Table 1 are shown comparisons of exact and approximate solutions for seven typical cases.

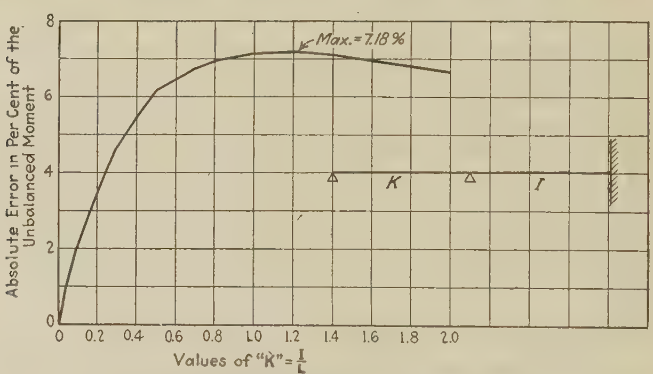


FIG. 3. CURVE SHOWING ERROR BETWEEN EXACT AND APPROXIMATE METHODS OF MOMENT DISTRIBUTION.

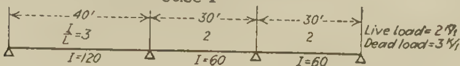
It will be noted that the errors are quite small. In general the relative error varies with the relative variations in span length and stiffness and with the relative intensity of the live load. Notice, further, that the proper basis of measurement of the relative error is the largest fixed end moment at the joint. The result cannot be more accurate than is this moment; no mathematics, no series of tests, no models can possibly eliminate this uncertainty as to bending moments in reinforced-concrete construction. It should be noted that the error in end moments may appear in the center moment as a somewhat larger relative error.

EFFECT OF HAUNCHES

Haunches at the ends of a beam increase the fixed end moments. They also affect the distribution factors—the relative stiffness of the members—and also the carry-over factors—the proportion of the distributed moment which is carried to the next joint. But we have already seen that the relative distribution is less important than the fixed end

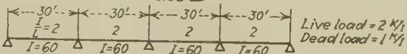
TABLE 1 - MAXIMUM POSITIVE AND NEGATIVE MOMENT VALUES BY THE EXACT AND APPROXIMATE METHOD OF MOMENT DISTRIBUTION

Case I



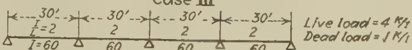
		C.L.		C.L.		C.L.		
Max.-M @ Support	0		-630.5		-427.5		0	Approx.
	0		-660.5		-451.5		0	Exact
Max.+M @ C.L. Span		+757.9		+169.8		+410.0		Approx.
		+747.5		+158.6		+400.8		Exact

Case II



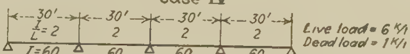
		C.L.		C.L.		C.L.		C.L.		
Max.-M @ Support	0		-308.4		-253.9		-308.9		0	Approx.
	0		-312.4		-257.3		-312.4		0	Exact
Max.+M @ C.L. Span		+246.2		+173.7		+173.7		+246.2		Approx.
		+240.8		+177.3		+177.3		+240.3		Exact

Case III



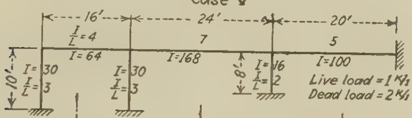
		C.L.		C.L.		C.L.		C.L.		
Max.-M @ Support	0		-521.4		-440.6		-521.4		0	Approx.
	0		-527.2		-449.8		-527.2		0	Exact
Max.+M @ C.L. Span		+426.0		+315.9		+315.9		+426.0		Approx.
		+424.5		+323.1		+323.1		+424.5		Exact

Case IV



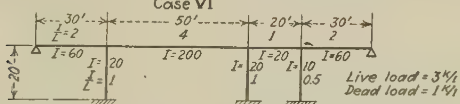
		C.L.		C.L.		C.L.		C.L.		
Max.-M @ Support	0		-734.7		-628.0		-734.7		0	Approx.
	0		-746.8		-642.5		-746.8		0	Exact
Max.+M @ C.L. Span		+606.4		+457.7		+457.7		+606.4		Approx
		+594.7		+466.1		+466.1		+594.7		Exact

Case V



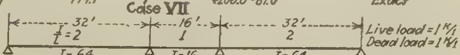
		C.L.		C.L.		C.L.		
Max.-M @ Support	-25.2		-105.0 -122.4		-133.5 -123.9		-100.4	Approx
	-26.4		-104.4 -124.5		-134.2 -124.9		-99.5	Exact
Max.+M @ C.L. Span		+40.7		+103.2		+50.5		Approx.
		+40.1		+101.3		+50.9		Exact
Max. M Top Column	-25.2		-23.3		-14.4			Approx.
	-26.4		-26.3		-14.3			Exact

Case VI



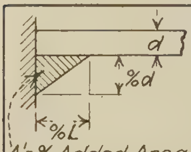
		C.L.		C.L.		C.L.	C.L.		
Max.-M @ Support	0		-642.0 -736.7		-437.1 -286.5 -222.6 -268.4			0	Approx.
	0		-627.2 -743.9		-468.8 -295.6 -235.4 -289.1			0	Exact
Max.+M @ Support	0		-147.0 -163.8		-28.9 -1.1 +4.1 -19.2			0	Approx.
	0		-144.0 -165.2		-26.6 +6.6 +5.4 -24.4			0	Exact
Max.+M @ C.L. Span		+255.9		+813.8		+68.3	+368.2		Approx.
		+251.4		+796.5		+68.6	+354.7		Exact
Max.-M @ C.L. Span		-87.8		+65.0		+71.7	+50.6		Approx.
		-74.6		+63.8		+78.1	+51.1		Exact
Max. M Top Column			-143.2		+177.6 -71.3				Approx.
			-174.1		+206.0 -87.0				Exact

Case VII



		C.L.		C.L.		C.L.		
Max.-M @ Support	0		-103.6		-103.6		0	Approx.
	0		-112.5		-112.5		0	Exact
Max.+M @ C.L. Span		+219.4		+33.6		+219.4		Approx.
		+215.7		+32.0		+215.7		Exact
Max.-M @ C.L. Span		+97.6		-4.6		+97.6		Approx.
		+95.9		-8.3		+95.9		Exact

TABLE 2—COMPARISON OF EXACT AND APPROXIMATE CONSTANTS FOR HAUNCHES AND VARIATION OF FIXED-ENDED MOMENTS FOR UNIFORM LOAD IN PER CENT OF THE FIXED-ENDED MOMENT FOR A PRISMATIC BEAM



%L	%d	%A'	Haunched One End Only: Haunched End	Haunched One End Only: Un-Haunched End	Haunched Both Ends	Method	Type of Haunch
15	19	2.8	105	97	105 103	Exact Approx.	Straight
		1.9	104	98	104 102	Exact Approx.	Parabolic
	49	7.4	115	93	110 107	Exact Approx.	Straight
		4.9	110	95	108 105	Exact Approx.	Parabolic
	71	10.7	121	89	113 111	Exact Approx.	Straight
		7.1	114	93	110 107	Exact Approx.	Parabolic
	102	15.3	131	85	115 115	Exact Approx.	Straight
		10.2	120	90	112 110	Exact Approx	Parabolic
20	19	3.8	113 108	95 96	106 104	Exact Approx.	Straight
		2.5	109 105	96 97	104 103	Exact Approx.	Parabolic
	49	9.8	125 120	88 90	113 110	Exact Approx.	Straight
		8.5	120 117	91 91	110 109	Exact Approx.	Parabolic
	71	14.2	133 128	84 86	116 114	Exact Approx.	Straight
		9.5	126 119	88 90	112 110	Exact Approx.	Parabolic
	102	20.4	144 141	81 80	119 120	Exact Approx.	Straight
		13.6	132 127	86 86	115 114	Exact Approx.	Parabolic
25	19	4.8	113 110	94 95	107 105	Exact Approx.	Straight
		3.2	109 106	95 97	105 103	Exact Approx.	Parabolic
	49	11.3	131 123	87 89	115 111	Exact Approx.	Straight
		7.5	122 115	88 92	111 108	Exact Approx.	Parabolic
	71	17.8	141 136	82 82	118 118	Exact Approx	Straight
		11.9	131 124	86 86	114 112	Exact Approx.	Parabolic
	102	25.5	151 151	78 74	122 126	Exact Approx.	Straight
		17.1	139 134	83 83	118 117	Exact Approx.	Parabolic

moment and evidently the proportion which is carried over is less important than the proportionate distribution. The writer has found it surprisingly satisfactory to correct the fixed end moments for the effect of haunches, and otherwise to neglect the haunching. This will not be true in the case of very large haunches but very large haunches are rarely justified and if the case be important more exact studies can be made.

There is a remarkable relation between the increase in end moment due to haunching and the increase in area of the side elevation of the beam. At a haunched end the fixed end moment is increased relatively about as much as the area of the side elevation of that half of the beam is increased and at the other end the fixed end moment is decreased

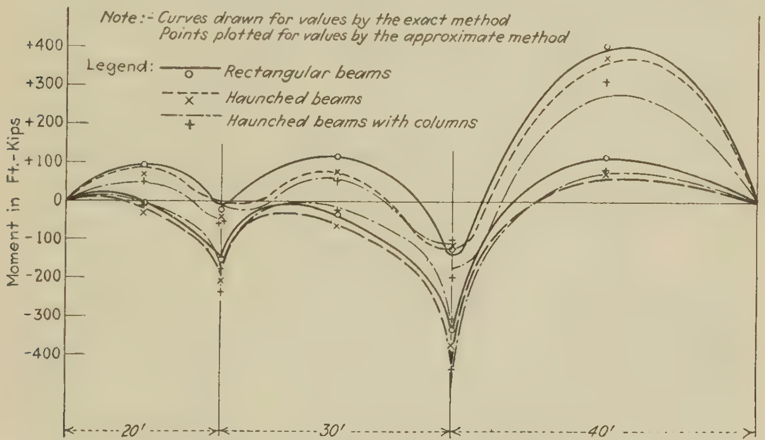


FIG. 4. CURVES OF MAXIMA—COMPARISON OF THE EXACT AND APPROXIMATE METHODS OF MOMENT DISTRIBUTION AND SHOWING EFFECTS OF HAUNCHING AND COLUMNS ON THE MOMENTS.

about half as much relatively as this increase in area. The effects of haunching both ends are additive. Table II gives data to support this approximation.

Fig. 4 gives a comparison in one case of results by this approximate method with those by "exact" analysis. The writer has on file many other comparisons, but a discussion of and interpretation of all these would involve mathematical relations not here presented. He does not wish to present the above as a general rule for all extremes of haunching, but in the usual case, where span lengths do not differ too greatly and haunches are not very large and are not dissimilar, the rule will make reasonable allowance for haunching.[1]

[1] The writer has discussed the exact analysis of haunched beams in a paper dealing with the general theory of moment distribution. This paper is now in the hands of another technical society. It was submitted in February, 1928, but at this writing (December), their committee on meetings and publications has not passed on it; for this reason, as well as for brevity, the treatment of haunched beams is here restricted to the approximate method.

SHEARS AND REACTIONS

Maximum shears have been neglected to a large extent in the literature of continuity. In case of equal spans maximum shear at the inner end of the end span may readily be 20 per cent greater than if the span were simply supported. A theorist may find even greater shears than this. Consider, for example, the first interior girder from the corner of the building. The continuity of beams and slabs will increase the load coming to this girder probably 20 per cent. The continuity of the girder with the next girder will increase the shear at the first interior column by another 20 per cent, and the maximum shear at the end of this girder may be 40 or 50 per cent greater than if there were no continuity in the frame work. The writer does not know why the increase in shear in the

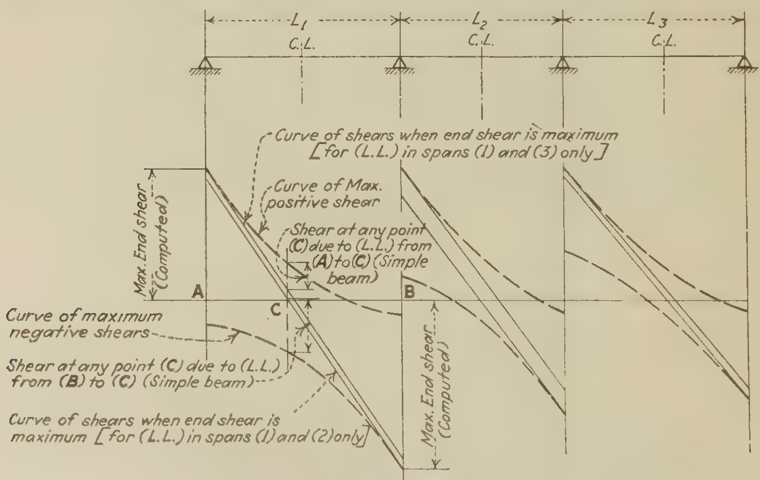


FIG. 5. TYPICAL CURVES OF MAXIMUM SHEARS (APPROXIMATE) FOR UNIFORM LOAD.

end span was neglected and the increase in moment so carefully defined when the conventional rules were first stated.

Maximum end shear will occur for the same loading that gives maximum end moment in a girder. The end moments may be taken from the tabulations already made for moment computations. The difference between the moments at the two ends of a span divided by the span length represents the change in end shear from that which would exist were the span simply supported. If the negative moment is greater at that end of the span than at the other, the shear at that end will be increased. Computations for maximum end shear seem worth making, though not with great precision.

Curves of maximum shear in a span are tedious to determine because the maxima require different partial loadings of the span in question. The writer has found that a very satisfactory rule for drawing these

curves is to add to the curves of shears which exists in the span when the end shear is maximum, the shear at the point in question due to live loads between this point and the end, computed on a simple span. The construction is shown in Fig. 5. As indicated later, however, it is not quite clear how to use these curves when they are available, though it is certainly important to have some idea of their shape.

Maximum reactions may be determined by adding maximum shears on two sides of a support to any loads which occur at this support.

ILLUSTRATION I. (SIMPLE CASE.)

The example here shown is for a simple case with uniform live and dead load and no columns. The values of I/L are first computed.

Live load is now applied in the first span and the fixed end moments computed $\frac{1}{12} wL^2 = \left(\frac{1}{12}\right) 2 \times 40 \times 40 = -267$. The free end is released and half the released moment brought over and added to give a total unbalanced moment of -400 at the first interior support.

This unbalanced moment is now distributed as follows:

$$\left(\frac{2}{2+3}\right)400 = -160.$$

Carry over, change sign, and distribute again $\left(\frac{160}{2}\right)\left(\frac{2}{2+2}\right) = +40$

For live load in the second span there are two unbalanced moments to distribute.

At left end $\left(\frac{3}{3+2}\right) 150 = -90$ to the left end span
$+60$ to the interior span.

Carry over, change sign, and distribute $\left(\frac{60}{2}\right)\left(\frac{2}{2+2}\right) = -15$

At the other end $\left(\frac{2}{2+2}\right)150 = -75$ to the right end span
$+75$ to the interior span

Carry over, change sign, and distribute $\left(\frac{75}{2}\right)\left(\frac{3}{2+3}\right) - 22.5$

Adding, total at left end of span $= -112.5$
at right end of span $= -\ 90.0$

The right end span is treated as was the left end.

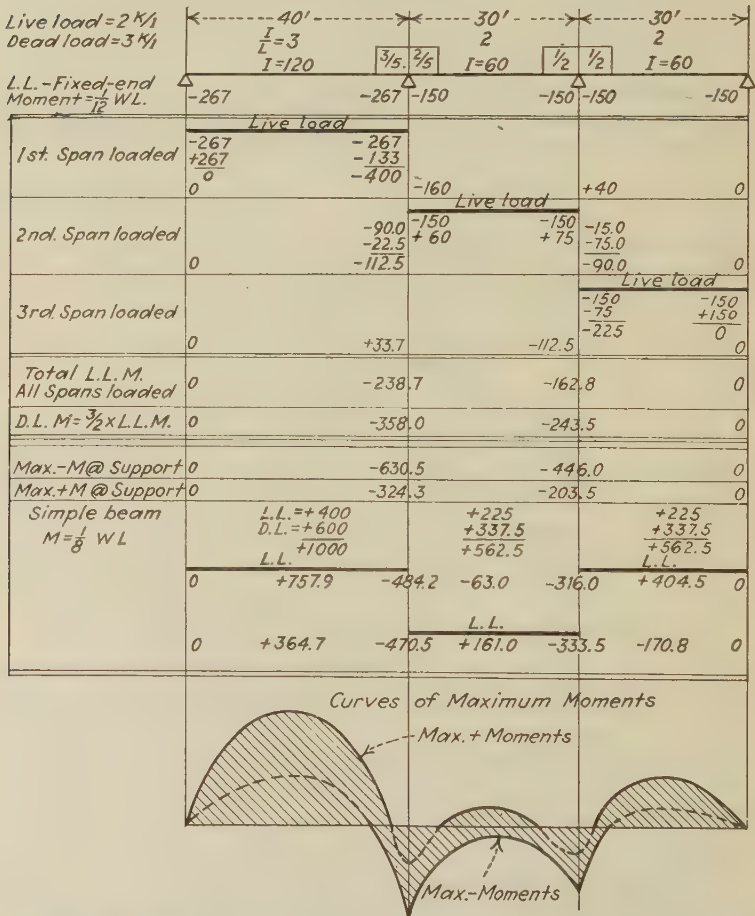
Adding all totals gives the moment at supports for uniform load over all girders of $2k$/ft. Multiplying by $\frac{3}{2}$ gives dead load moments at supports.

All negative moments at supports due to live load are now added to the corresponding dead load moments to give maximum negative moments

at supports. Similarly all positive live load moments at supports are added to the corresponding dead load moments to give maximum positive moments at supports. Due to the relatively great intensity of dead load, the maximum positive moments at supports (minimum negative moments) are both negative.

ILLUSTRATION I

THE APPROXIMATE METHOD FOR MAXIMUM MOMENTS



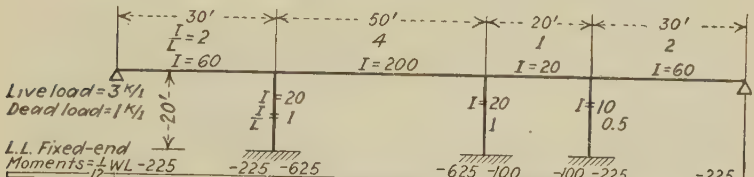
Now add dead load moments at supports to moments at supports for alternate spans loaded with live load beginning with the first span (in this example the left end span and the right end span) to get moments at the supports for this condition. The average moment at the ends added to the moment which would exist at center on a simple beam for

the corresponding loads (live and dead in end spans, and dead only in center span) gives maximum moments at centers of the spans. These are positive maxima in the end spans and negative maximum in the center span.

Similar computations are made for live load in the center span (alternate spans beginning with the second.)

ILLUSTRATION II

THE APPROXIMATE METHOD FOR MAXIMUM MOMENTS



1st. Span loaded	Live load -225 +225 0 0	-225.0 -112.5 -337.5 +96.2 -241.3	-192.8	+32.1	+16.1	-5.7	-4.6	0
2nd. Span loaded	0	-178.8 -59.5 -238.3	Live load -625 +357.5 -89.4 -356.9	-625 -59.6 +41.7 -267.6	-29.8 -104.0 -133.8	+10.6 +37.2 +47.8	+8.5 +29.8 +38.3	0
3rd. Span loaded	0	+9.5 +1.3 +10.8	+14.3 +2.0 +16.3	-66.8 -9.5 -76.3	Live load -100 +16.7 -11.9 -95.2	-100 -5.9 +28.6 -77.3	-4.8 -57.0 -61.8	0
4th. Span loaded	0	-4.6	-6.9	+32.2	+40.3	-96.5	Live load -225 -112.5 -337.5 +193.0 -144.5	-225 +225 0 0
Total L.L.M. All Spans loaded	0	-473.4	-540.3	-279.6	-172.6	-131.7	-172.6	0
D.L.M. = 1/3 × L.L.M.	0	-157.8	-180.1	-93.2	-57.5	-43.9	-57.5	0
Max. −M @ Support	0	-642.0	-736.7	-437.1	-286.5	-223.4	-268.4	0
Max. +M @ Support	0	-147.0	-163.8	-28.9	-1.1	+3.9	-19.2	0
Simple beam M = 1/8 WL	L.L. = +337.5 D.L. = +112.5 +450.0		+937.5 +312.5 +1250.0		+150 +50 +200		+337.5 +112.5 +450.0	
	L.L. 0 +255.9	-388.3	-356.6 +65.0	-137.4	L.L. -136.6 +68.3	-126.9	-123.9 +50.6	0
	0 -87.8	-400.7	L.L. -543.9 +813.8	-328.6	-71.7 -151.0	-92.4	L.L. -163.7 +368.2	

Note. Moments are for girders

The curves of maximum moments are shown but details of their construction are omitted. (See Figs. 1 and 2.)

Maximum shears and reactions have not been computed for this case.

ILLUSTRATION II. (WITH COLUMNS.)

The procedure is the same as in the first illustration. Different girder moments, however, must be carried on two sides of the column at

any support. The details of the distribution may need a little more explanation.

Consider live load in the second span. Fixed end moments are $\left(\frac{1}{12}\right)3 \times 50^2 = -625$. Distributing at left end gives $\left(\frac{2}{2+1+4}\right)625 =$ -178.8 to first span and $\left(\frac{4}{2+1+4}\right)625 = +357.5$ to second span.

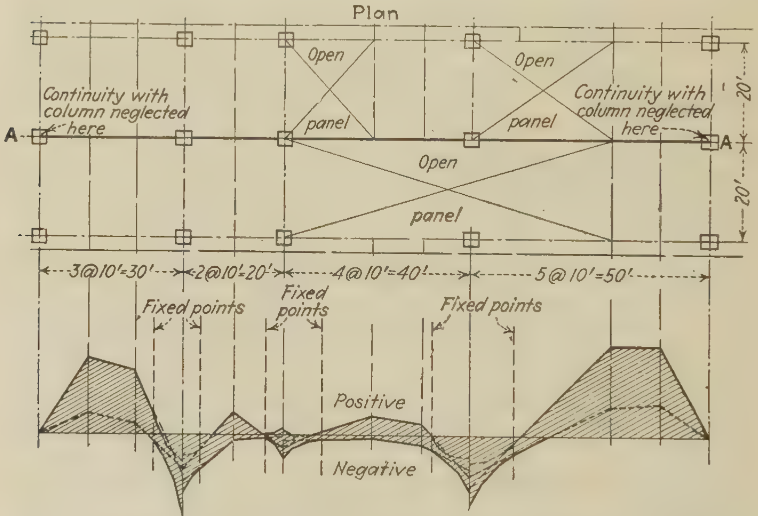


FIG. 6. CURVES OF MAXIMUM MOMENTS FOR GIRDES ON LINE *AA* IN A PLATFORM CONTAINING OPEN PANELS AS SHOWN

Change sign, carry over and distribute

$$\left(\frac{357.5}{2}\right)\left(\frac{1+1}{4+1+1}\right) = -59.6 \text{ in second span (right end)}$$

$$\left(\frac{357.5}{2}\right)\left(\frac{1}{4+1+1}\right) = -29.8 \text{ in third span (left end)}$$

Change sign, carry over, and distribute

$$\left(\frac{29.8}{2}\right)\left(\frac{2+0.5}{1+2+0.5}\right) = +10.6 \text{ in third span (right end)}$$

$$\left(\frac{29.8}{2}\right)\left(\frac{2}{1+2+0.5}\right) = +8.5 \text{ in fourth span (left end).}$$

Since the procedure is approximate it may as well be done on the upper scale of the slide rule and need never be taken out of the slide rule till all distributions in one direction have been made.

Note that signs are such as will balance the joint—that is, will make the total values the same on both sides in value and in sign. The column moments, however, are not written until the end, when they may be found by subtraction, if needed. They have not been computed here.

ILLUSTRATION III APPROXIMATE METHOD

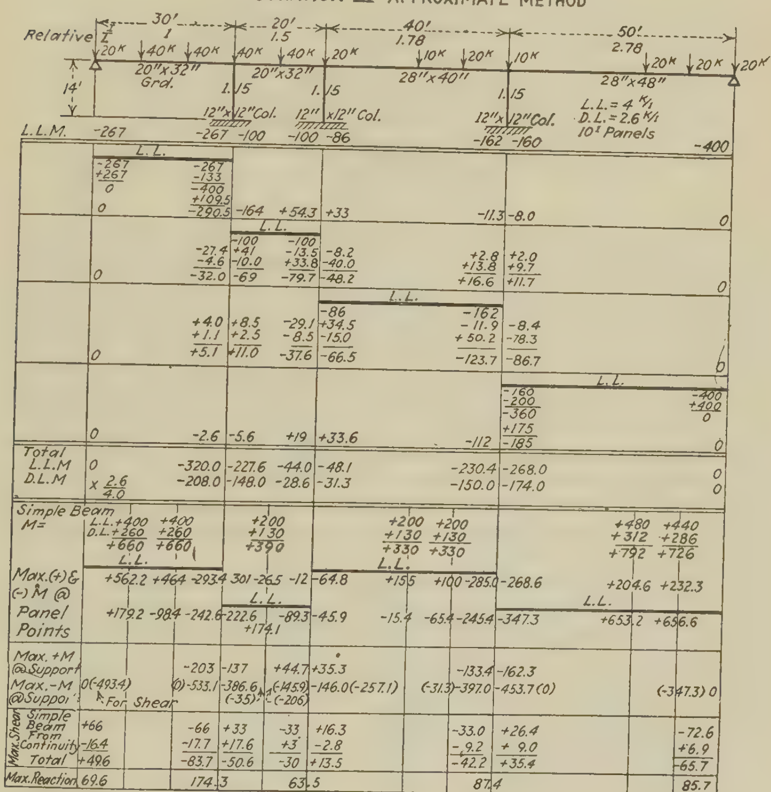


ILLUSTRATION III. (IRREGULAR LOADING.)

This represents a girder in a platform having blank panels as shown in Fig. 6. All dead load has for simplicity been assumed as concentrated at panel points. The outer ends of the girders have been treated as freely supported.

The computation of fixed end moments deserves notice.

$$\text{In the first span } 40\left(\frac{20 \times 10}{30}\right)\left(\frac{2}{3}+\frac{1}{3}\right) = -267$$

$$\text{In the second span } 40 \times \frac{20}{4} \times \frac{1}{2} = -100$$

$$\text{In the third span } 20 \times \left(\frac{10 \times 30}{40}\right)\left(\frac{1}{4}\right) = -36 \times 3 = -112$$

$$10 \times \left(\frac{40}{4}\right)\left(\frac{1}{2}\right) = \qquad \frac{-50}{-86} \qquad \frac{-50}{-162}$$

$$\text{In the fourth span } 40 \times \left(\frac{10 \times 40}{50}\right)\left(\frac{1}{5}\right) = -64 \times 4 = -256$$

$$20 \times \left(\frac{20 \times 30}{50}\right)\left(\frac{2}{5}\right) = \frac{-96}{-160} \times \frac{3}{2} = \frac{-144}{-400}$$

The procedure for moment computations follows that given in the two cases above. It is interesting, though perhaps not especially important, to note that the curves of maximum moment are not straight lines from the support to the first beam but break at the fixed points and once between fixed point and support for each panel point in the span. The exact location of points on this curve is, however, rarely worth while.

The computation of maximum shears and reactions deserves attention. When the maximum negative moments at supports are computed the moments at the other ends of the beams for the same loading are also computed and written in brackets. For the end support, since there is no moment, it is necessary to understand that live loads should be in the end span and alternate spans beyond; moments for this case have already been computed.

The end shears are now computed assuming the spans to act as simple beams. These are changed by the quotient of the change in end moment divided by the span. When the negative moment is greater at the end considered, the shear is increased at that end.

The sum of the maximum shears at any support plus loads over the support gives the maximum reaction.

Thus in this case the shear due to continuity is

$$\text{In the first span—Left end } \frac{493.4 - 0}{30} = 16.4 \text{ decrease}$$

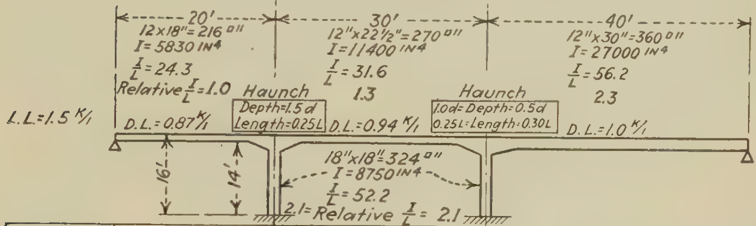
$$\text{Right end } \frac{533.1 - 0}{30} = 17.7 \text{ increase}$$

$$\text{In the second span—Left end } \frac{386.6 - 35}{20} = 17.6 \text{ increase}$$

$$\text{Right end } \frac{206.0 - 145.9}{20} = 3.0 \text{ decrease}$$

Curves of maximum shears have not been computed. They may be determined by finding shears for each individual load, or may be approximated as explained elsewhere.

ILLUSTRATION IV
MOMENT DISTRIBUTION BY THE APPROXIMATE METHOD
FOR CURVES OF MAXIMUM MOMENTS
WITH HAUNCHED BEAMS AND WITH COLUMNS



L.L.M=½ W.L. Effect due to haunches	-50 +18.8=	Added area 37.5% x2=	-50 37.5	-112.5 -84.6=2x +28.1=	Added area 37.5% = 25% x2=	-112.5 +42.3 -56.2	-200 -60 = 2x	Added Area 15% =	-200 +30
Total L.L.M	-31.2		-87.5	-169.0		-125.4	-260		-170
1st. Span loaded	Live load -31.2 +31.2 0 0		-87.5 -15.6 -103.1 +23.4 -79.7	-30.4		+11.7	+6.1		0
2nd. Span loaded	0		-38.4 -3.3 -41.7	-169 +50 -10.1 -129.1	Live load	-126.4 -18.5 +28.8 -115.6	-97 -51.0 -60.7		0
3rd. Span loaded	0		+8.9	+27.7		-78.6	-260 -85 -345 +139 -206	Live load	-170 +170 0 0
Total L.L.M D L M	0 0	x 0.87/1.5	-112.5 -65.3	-131.8 -82.4	x 0.94/1.5	-182.5 -114.2	-260.6 -173.6	x 1/1.5	0 0
Max. -M @ support	0		-186.7	-241.9		-308.4	-440.3		0
Max. +M @ support	0		-56.4	-54.7		-102.5	-200.0		0
Simple beam M=⅛ WL		L.L.=+75 D.L.=+43.5 +118.5			+168.8 +105.7 +274.5			+300 +200 +500	
		L. L.						L. L.	
	0	+50.5	-136.1	-85.1	-27.4	-181.1	-373.5	+313.3	0
					L. L.				
	0	-10.0	-107.0	-211.5	+53.9	-229.8	-234.3	+82.9	0
Values for Max. M Curves	+M -M	C.L. +50.5 -10.0	-56.4 -186.7	-54.7 -241.9	C.L. +53.9 -27.4	-102.5 -308.4	-200.0 -440.3	C.L. +313.3 +82.9	

ILLUSTRATION IV. (HAUNCHED BEAMS.)

This problem illustrates the application of the approximate method of allowing for the effect of haunches. The solution is not correct as regards the columns, no allowance having been made for the changed stiffness of columns produced by flare at their tops.

The fixed end moments are first computed as if the girders were not haunched. For each haunch the moment is increased at the haunched end by the same percentage as the side elevation of that half of the girder is increased by the haunching and at the other end is decreased by half this percentage.

TABLE 3 - VALUES FOR CURVES OF MAXIMA IN (FIG. 4) BY THE EXACT AND APPROXIMATE METHOD OF MOMENT DISTRIBUTION

		20′ C.L.			30′ C.L.			40′ C.L.
Case I Rectangular beams								
+M	Exact	+93.6	-16.4		+116.0	-134.0		+399.0
+M	Approx.	+91.5	-23.1		+115.0	-124.7		+406.2
-M	Exact	-5.2	-148.0		-40.9	-352.0		+112.0
-M	Approx.	-8.3	-152.5		-36.7	-339.6		+113.7
Case II Haunched beams								
+M	Exact	+84.0	-2.1		+72.2	-126.9		+363.0
+M	Approx.	+68.7	-41.5		+73.0	-121.2		+377.2
-M	Exact	-19.5	-192.7		-65.7	-426.5		+60.4
-M	Approx.	-32.4	-210.1		-67.0	-375.6		+74.5
Case III Haunched beams with columns								
+M	Exact	+49.3	-48.9	-10	+59.3	-111.4	-183.5	+279.5
+M	Approx.	+50.5	-56.4	-54.7	+53.9	-102.5	-200.0	+313.3
-M	Exact	-4.5	-186.0	-206	-19.2	-352.0	-507.0	+72.3
-M	Approx.	-10.0	-186.7	-241.9	-27.4	-308.4	-440.3	+82.9

Thus, in the second span, the fixed end moment for a prismatic beam is $\left(\frac{1}{12}\right)1.5 \times 30^2 = -112.5$. The effect of the haunch at the left end, which has a depth 1.5 times as great as at the center (total depth at end 2.5 times center depth) and extends to the quarter point of the span is to increase the area of side elevation of this end of the beam 75 per cent. The fixed end moment at this end is therefore increased 75 per cent of 112.5 = 84.6 and that at the other end is decreased 42.3. Allowance is made in the same way for the haunch at the right end of the beam. The effects of both haunches are then added to the moment originally computed.

The distribution of unbalanced moments at supports and combination of these moments to give maximum moments then follows the procedure for beams of uniform section, the I values being computed for the depth at the center.

Fig. 4 shows a comparison of maxima computed for this case by this method with curves of maxima arrived at by theoretically exact analyses. It indicates also the effect on maximum moments of haunching and of columns. The same data are given in tabular form in Table 3.

ILLUSTRATION V APPROXIMATE METHOD

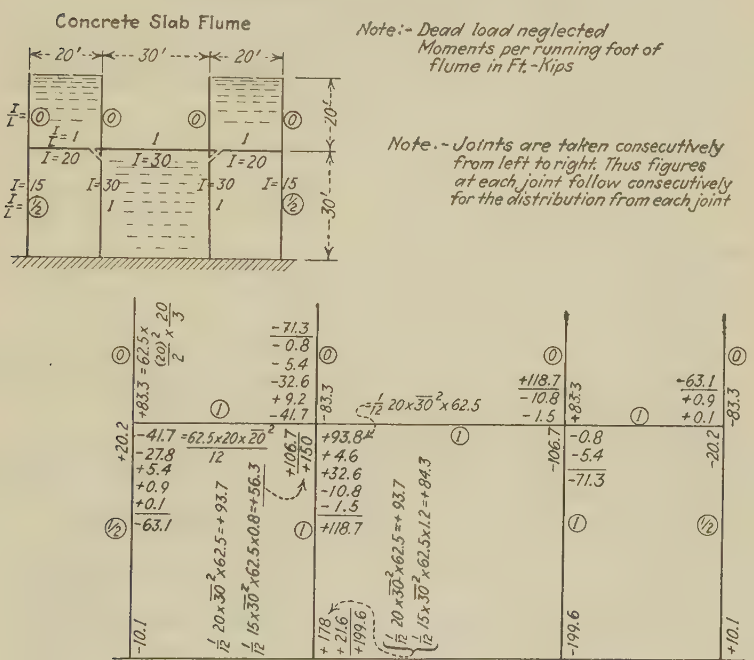


ILLUSTRATION V. (SIGN CONVENTIONS.)

The structure shown here represents a series of water conduits having continuous walls and bottom slabs, the upper and lower reservoirs being connected so that the lower reservoir is under a static head. The problem is devised to bring out clearly the convention of signs used.

Use the usual moment conventions on both girders and columns looking at them as a drawing is usually read.

Write girder moments parallel to the girder. In this case the girder moments are written below the girder at the right end and above at the left end in order to avoid interference with the rows of figures for the

columns. Usually it makes no difference whether girder moments are written above or below the girders.

Write column moments parallel to the columns, above the column at the top and below at the bottom. This is an essential part of the sign convention used.

A joint will be balanced when the total moment to left and to right of the joint (the sheet being held in its usual position) are the same both in value and in sign.

The fixed end moments are first written for the vertical walls and for the girders. The first projecting wall on the left is bent and the moment is +83.3; the wall of the bottom tank is bent at its ends and the moment is

due to uniform load

$\left(\frac{1}{12}\right)20 \times 62.5 \times 30^2 =$	93.7 at top	93.7 at bottom

due to triangular load [1]

$\left(\frac{1}{12}\right)15 \times 62.5 \times 30^2 \times \frac{8}{10} =$	56.3 × 1.5	84.3 at bottom
Total......................	+150.0 at top	+178.0 at bottom

In the girders the end moments are −41.7 in the end girder and +93.8 in the interior girder.

Unbalanced moments are distributed in succession beginning at the left end. The moments distributed to girders only are written and column moments found later by balancing joints.

At the left end +83.3 − 41.7 = +41.6 is unbalanced. Girder takes $\left(\frac{1}{1 + \frac{1}{2}}\right)41.6 = 27.8$, the stiffness of the vertical projecting wall being zero. The moment to the girder is negative because this tends to reduce the positive moment on that side of the joint and so balance the joint.

Carry over and change sign $\left(\frac{1+1}{1+1+1}\right)\left(\frac{27.8}{2}\right) = +9.2$ and $\left(\frac{1}{1+1+1}\right)\left(\frac{27.8}{2}\right) = +4.6$.

Carry over and change sign $\left(\frac{1}{1+1+1}\right)\left(\frac{4.6}{2}\right) = -1.5$ and $\left(\frac{1}{1+1+1}\right)\left(\frac{4.6}{2}\right) = -0.8$

[1] Fixed end moments with triangular loading are 20 per cent greater at one end and 20 per cent less at the other than if the load were uniformly distributed.

Carry over and change sign $\left(\dfrac{0.5}{1 + 0.5}\right)\left(\dfrac{0.8}{2}\right) = +0.1$

Distributing unbalanced moments at the second joint, we have

$$\left.\begin{matrix}-41.7\\+150.0\end{matrix}\right) \text{ on the left}$$

$$\left.\begin{matrix}-83.3\\+93.8\end{matrix}\right) \text{on the right}$$

Total $+97.8$ unbalanced on the left.

Each girder takes $\left(\dfrac{1}{3}\right)\left(97.8\right) = 32.6$. On the left this is negative, on the right positive to balance the joint. These moments are then distributed, thus

$$+5.4 \longleftarrow -32.6 \;\Big|\; +32.6 \longrightarrow -10.8 \;\Big|\; -5.4 \longrightarrow +0.9$$

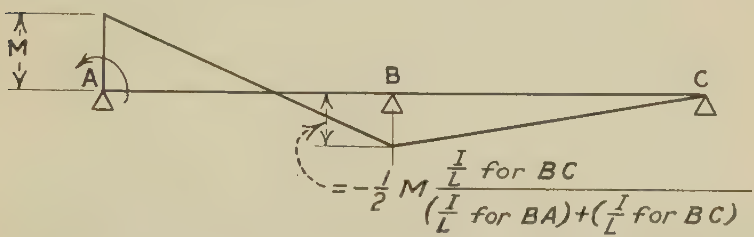


FIG. 7. MOMENT CURVE OF TWO-SPAN BEAM SHOWING DISTRIBUTION BETWEEN SPANS.

Distributions from the third and fourth joints are, of course, as above. Addition now gives total end moments in the girders.

Moments in the columns are now found by balancing the joints. At the left joint, $\left.\begin{matrix}+83.3\\-63.1\end{matrix}\right)$ on the right is balanced by $+20.2$ written on the left of the column. At the next joint $\left.\begin{matrix}-83.3\\+118.7\end{matrix}\right)$ on the right and -71.3 on the left are balanced by $+106.7$ in the column (total each side $+35.4$). This is a change of $106.7 - 150.0 = -43.3$ in the column, half of which is carried to the bottom to give a total there of $+199.6$.

EXACT METHOD OF ANALYSIS

Many engineers will wish a convenient exact method as a basis for comparison with the approximate method shown.

Consider any two-span beam ABC, as shown in Fig. 7. Apply at A a moment M. Assume B held rigidly against rotation. The moment

at B is now $-\frac{M}{2}$. Now let joint B rotate. The moment at B is distributed between BA and BC in proportion to their I/L values, the resulting total moment at B being

$$-\tfrac{1}{2}M \frac{I/L \text{ for BC}}{\frac{I}{L}\text{for BA} + \frac{I}{L}\text{for BC}}.$$

The final moment curve on the beam is now as shown in the figure.

The rotation at A is $\frac{1}{EI}$ times the shear at A due to this moment curve as a load on AB.

$$\text{Rotation} = \tfrac{1}{3}M\frac{L}{EI}\left(1 - \tfrac{1}{4}\frac{I/L \text{ for BC}}{\frac{I}{L}\text{ for BA} + \frac{I}{L}\text{ for BC}}\right)$$

Hence the moment needed to produce unit rotation at A is proportional to

$$\frac{I}{L}\text{for BA}\frac{1}{1 - \tfrac{1}{4}\dfrac{1}{1 + \dfrac{I/L \text{ for BA}}{I/L \text{ for BC}}}} = \frac{I/L \text{ for BA}}{C}$$

Values of the function $(\frac{1}{C} - 1)$ expressed as percentage increase for various ratios of the I/L values are shown in Fig. 8. The values have been plotted in terms of the ratio of the larger to the smaller I/L in order to increase the accuracy for small values of $\frac{I/L \text{ for BA}}{I/L \text{ for BC}}$

If there are several connecting members at B, some of which are stiffened by continuity at their far ends, I/L for BC will be replaced by $\Sigma\frac{I/L}{C}$ of all connecting members. The distribution at B is in the ratio of I/L for BA to $\Sigma\frac{I/L}{C}$ for all members such as BC.

This gives an exact technique for "precise" analysis. Starting at one end of the series, write $\frac{I/L}{C}$ values—adjusted I/L values—for the far end of each member to the other end of the series. The values of 1/C may be computed or taken from the curve.

At any joint distribute the unbalanced moment in proportion to the adjusted I/L values of connecting members. Carry over one half the moment distributed to any beam and at the other end of the beam distribute it in the ratio of the I/L of this beam—not the adjusted value—and the adjusted values of I/L for connecting beams.

Unbalanced moments at successive joints are thus distributed and all distributed moments added to the original fixed end moment, just as in the approximate method.

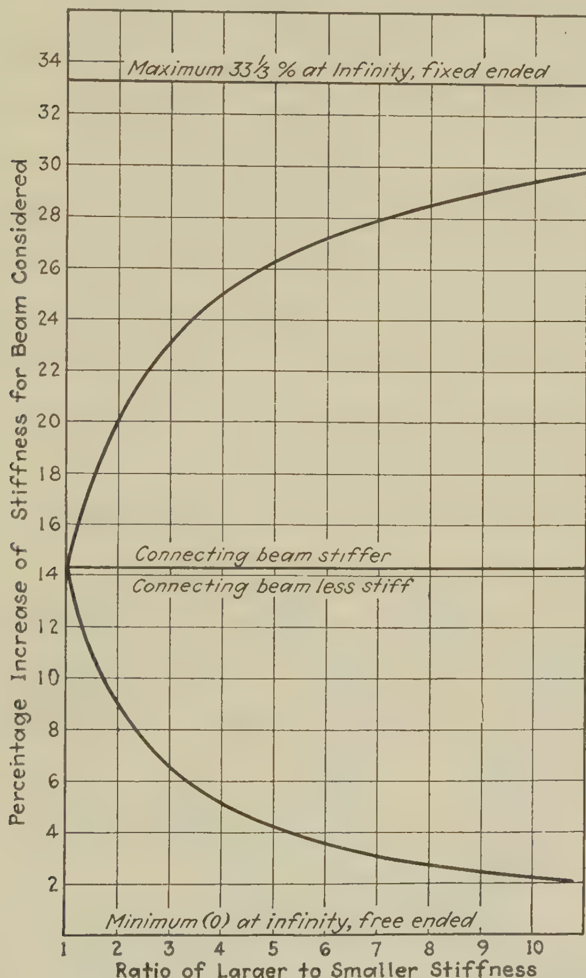


FIG. 8. CURVE GIVING PERCENTAGE INCREASE IN STIFFNESS OF BEAM DUE TO CONNECTING MEMBERS.

ILLUSTRATION VI.—EXACT METHOD.

The following procedure illustrates the process just explained. For simplicity, an unbalanced moment is assumed to exist at one joint only. All figures are shown. Most of the computation would be done mentally

and need not be very accurate. It is important to note that in distributing moments which are carried over in a member, the I/L for that member and not its adjusted I/L is used. For other members connecting at that joint use the adjusted I/L values.

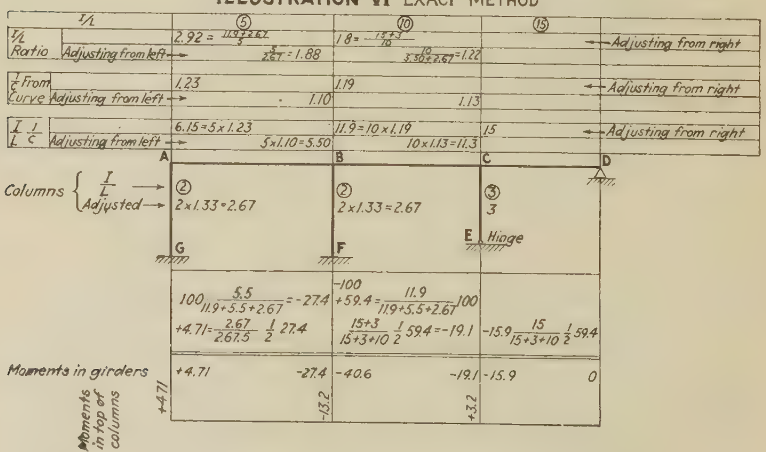
Beginning at the right end; member CD is hinged at the far end and its I/L value needs no adjustment; member CE is hinged and its I/L value needs no adjustment; for member BC, sum of I/L of connecting members at C = 15 + 3 = 18, ratio of I/L values $\frac{18}{10} = 1.8$ from which $\frac{1}{C} = 1.19$ is found from the curve or by computing

$$\frac{1}{1 - \frac{1}{4}\cdot\dfrac{1}{1 + \dfrac{\text{I/L for member}}{\text{I/L for member} + \Sigma\left(\frac{I}{L}\,\frac{1}{C}\right)\text{for connected members}}}}$$

$$= \frac{1}{1 - \frac{1}{4}\,\dfrac{1}{1 + 1.8}} = 1.19,$$

then $10 \times 1.19 = 11.9$ for the adjusted I/L value of BC at B.

ILLUSTRATION VI EXACT METHOD



For AB, $\Sigma\frac{I/L}{C}$ for connected members at B = 11.9 + 2.67, $\frac{I}{L}$ for member BF being adjusted to 2.67 because of fixation at F; ratio at

$A = \dfrac{11.9 + 2.67}{5} = 2.92$, for which curve gives $1/C = 1.23$ and $5 \times 1.23 = 6.15$ is the adjusted I/L value for AB at A.

All I/L values are now adjusted in the same way, working from the left end. For AB, I/L ratio is $\dfrac{5}{2.67} = 1.88$ which gives $1/C = 1.10$ (the connected members are less stiff than the main member) and the adjusted I/L value for AB at B is $5 \times 1.10 = 5.50$. Similarly the adjusted I/L value of BC at C is found to be 11.3.

Now distribute the unbalanced moment at B to the girders and column in the ratio of their adjusted I/L values at B (5.50 for AB, 11.9 for BC, 2.67 for BF). Do not write the column value. Carry over to joint C one-half of 59.4 and distribute there in the ratio of the I/L of BC and the adjusted values of I/L for CD and CE (in this case the adjusted values are the same as the I/L values because D and E are hinged). Of course the moment in CB equals the sum of the moments in CD + CE and so we can write at once for the moment in CB $\left(\dfrac{15 + 3}{10 + 15 + 3}\right)\left(\dfrac{59.4}{2}\right) = -19.1$ and then on the same slide rule setting the moment in CD $\left(\dfrac{15}{10 + 15 + 3}\right)\left(\dfrac{59.4}{2}\right) = -15.9$. Again the moment in the column is not written. Similarly the moment in BA is distributed to A.

Addition now gives the final moments in the girders. Moments in the columns are then found by subtraction.

Note that signs are nearly automatic because the signs at any joint must be such that the sum of the moments on two sides of the joint are alike both in sign and value. When moments are "carried over," they change signs.

ILLUSTRATION VII.

This shows another case in which the loads are fixed.

In actually using this method, the I/L ratios would be mentally figured and 1/C estimated. This is certainly all that is needed.[1]

SIDE SWAY AND HORIZONTAL LOADS

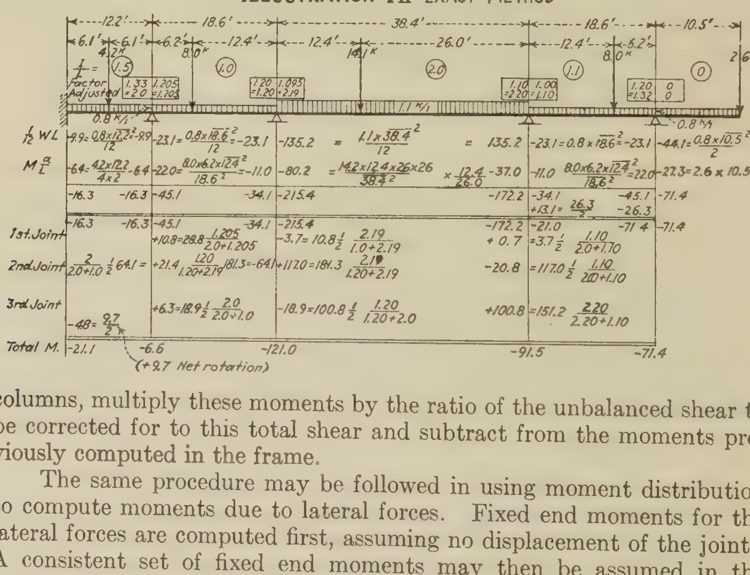
The theory indicated above is based on the assumption that the joints of the structure do not move. If the movement of the joints is due to settlement of foundations and can be predicted or determined in any way it can be allowed for as indicated below. A symmetrical frame symmetrically loaded with vertical loads has no tendency to displace its joints if the foundations do not settle. If it is restrained sideways its joints can not be displaced.

[1] A convenient approximation is to assume $1/C = 7/6$ for all interior spans.

If the frame is unsymmetrical or unsymmetrically loaded and subject to vertical loads it will be found that the solutions given above do not satisfy the laws of statics[1] because the sum of the shears on a horizontal section through the frame is not equal to zero. The frame will therefore move sideways until these shears are balanced. If there are several columns it will be found that this effect is negligible. In the case of isolated bents the effect may be worth evaluation.

The effect of this sidesway can be evaluated as follows: Assume a sidesway such as will produce a given consistent set of fixed end moments in the columns. If the columns are alike assume fixed end moments of 100 at each end of each column. If the columns are not alike the end moments should be proportional to $\frac{I}{L^2}$ for each column. Distribute these moments throughout the frame, compute the total shear existing in all

ILLUSTRATION VII EXACT METHOD



columns, multiply these moments by the ratio of the unbalanced shear to be corrected for to this total shear and subtract from the moments previously computed in the frame.

The same procedure may be followed in using moment distribution to compute moments due to lateral forces. Fixed end moments for the lateral forces are computed first, assuming no displacement of the joints. A consistent set of fixed end moments may then be assumed in the columns, distributed and the unbalanced shear produced by them computed as above. These moments may then be subtracted in proper proportion from the moments previously computed so as to satisfy the laws of statics.

This method will give a rapid, practical solution of many problems such as wind stresses in framed bents or traction stresses in viaducts. In nearly all cases, however, the girder is so relatively stiff compared with the column that its moment of inertia may be taken as infinite and fixed end moments in the columns therefore remain undistributed.

[1] Unless, by accident, the dissymmetry in loading balances the effect of unsymmetrical form.

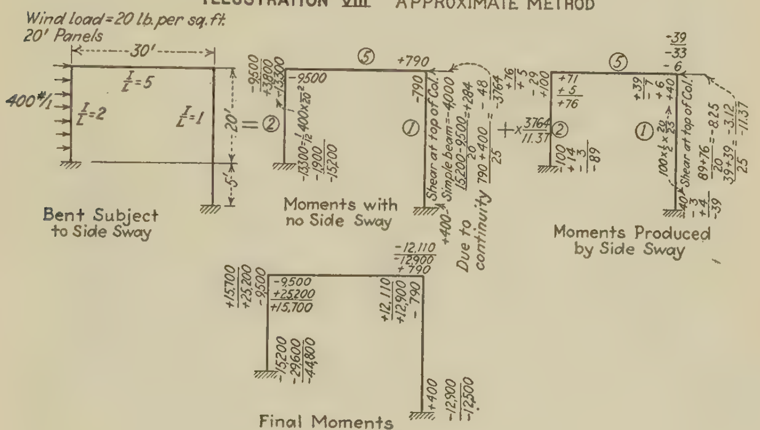
Illustration VIII. (Horizontal Loads)

This illustrates a wide range of cases, the precise computation of which is not usually of great importance.

In this case fixed end moments are first computed on the windward column and then distributed. This assumes that the joints do not move and hence the transverse shear at the top of the bent is unbalanced.

Now assume any consistent set of fixed end moments—that is, a set such that the fixed end moment in the columns is proportional to $\frac{I}{L^2}$, thus giving the same deflection at the top of each column. In this case assume ± 100 in the windward column and $\pm(\frac{1}{2})(\frac{20}{25})(100) = \pm 40$ in the leeward column. Distribute these moments.

ILLUSTRATION VIII APPROXIMATE METHOD



Now add the second set of moments to the first set in such proportion as to make the total transverse shear at the top of the bent equal to zero.

The method may be applied in exactly the same way no matter how many columns the bent has and no matter what the condition of transverse loading. It may be applied wherever the joints move.

The method may be extended to multiple story bents but the solution must be made carefully and simultaneous equations are needed to balance the shears. Exact solutions in such cases are usually of academic rather than practical interest.

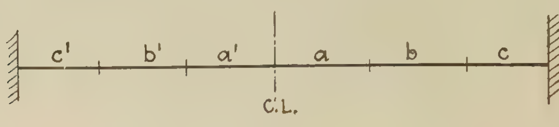
Column Moments and Their Effects

The effect of the girders on the columns is much greater than the effect of the columns on the girders. Moments in the girders are the sum of the fixed end moments and the distributed moments. Fixed end

moment, though subject to some uncertainty as explained above, is pretty definitely known. The column simply affects the distribution of the unbalanced moment and this unbalanced moment is only a part, and often a small part, of the total moment in the member.

But in general, the column has no transverse load and no fixed end moment and therefore the only moment in it is the distributed moment

TABLE 4 – VARIATION IN END MOMENTS OF A FIXED-ENDED BEAM FOR A VARIATION OF THE PHYSICAL PROPERTIES. (E) decreased ($33\frac{1}{3}$ %) for designated Sections



For Full Uniform Load — Normal −18.0

M	% Variation	Sections Varied	% Variation.	M
-19.04	+6	a'b'b	+3	-18.55
-20.09	+12	a'b'abc	-4	-17.21
-20.12	+12	a'b'ac	-6	-16.94
-19.79	+10	a'b'bc	-8	-16.58
-16.72	-7	a'c'b	+8	-19.36
-16.94	-6	c'ac	-3	-17.45
-19.15	+6	a'ab	+7	-19.22
-17.23	-4	a'b'c'ab	+12	-20.11
-16.94	-6	a'c'ab	+12	-20.12
-17.04	-5	a'b'c'b	+7	-19.34
-19.53	+9	abc	-7	-16.75
For Unit Load at Center of Span				Normal −1.50
-1.60	+7	a'b'b	+3	-1.54
-1.397	-7	a'c'ab	+16	-1.735
-1.65	+10	a'ab	+7	-1.61
-1.39	-7	a'b'c'ab	+16	-1.75
-1.40	-7	c'ac	-3	-1.46
-1.37	-9	a'b'c'b	+10	-1.65
-1.36	-9	a'c'b	+10	-1.65
-1.74	+16	a'b'abc	-7	-1.39
-1.74	+16	a'b'ac	-7	-1.40
-1.715	+14	a'b'bc	-13	-1.30
-1.67	+11	abc	-9	-1.36

which comes to it. There is good reason, as shown later, to believe that the moments in the girders can be determined with reasonable accuracy but there seems equally good reason to doubt whether the moments in the columns can be determined with much accuracy. Nevertheless an effort should be made to include the column effects in the analysis.

ACCURACY OF GIRDER MOMENTS

The chief element affecting the end moments in the girders is the original fixed end moment. Now it is certain that the moment of inertia of a reinforced concrete girder varies along its length. At the center it normally acts as a T beam. At the end the T is on the tension side of the beam and while probably effective to some extent even up to failure, is not as effective as when in compression.

It becomes important, then, to ask whether uncertainty as to the variation of (EI) in a concrete girder arising from variation of the quality

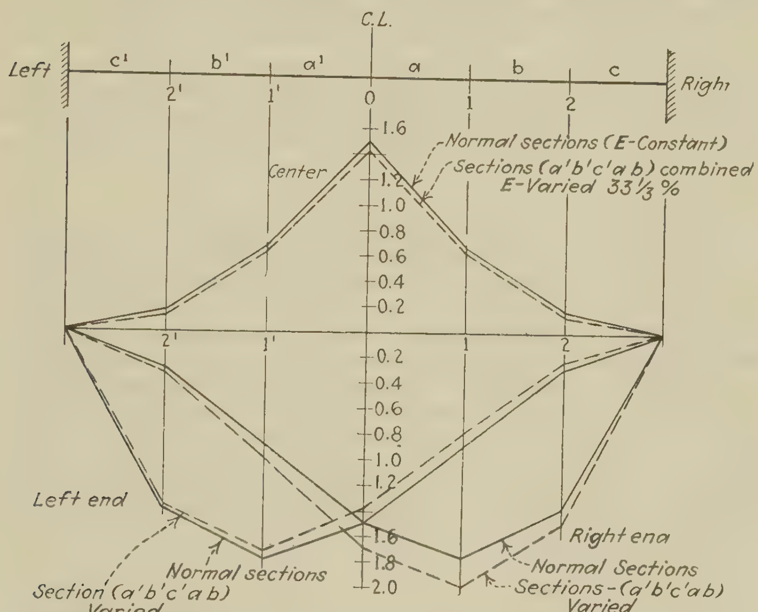


FIG. 9. VARIATION IN THE INFLUENCE LINES FOR MOMENT AT CENTER AND ENDS FOR A DECREASE OF THE MODULUS OF ELASTICITY OF 33⅓ PER CENT FOR SECTIONS INDICATED

of the concrete or from uncertainty as to the physical action of the sections may not seriously affect the computed fixed end moments. This question can be studied in a semi-empirical way by assuming[7] different combinations of the EI values along the girder section and computing fixed end moments. After assuming properties for the girder we apply principles which are purely geometrical and in no way involve any theory of structural action.

Table 4 shows the result of such a study. A straight beam has been divided into six sections and some of these sections have been assumed one third stiffer than others. For certain selected combinations of stiff

and flexible sections fixed end moments have been computed for a load at the center of the span and for uniform load. For certain cases influence lines are shown in Fig. 9. It will be seen that the error in the fixed end moment is not very large, but is not entirely negligible. It should be said that some of the cases taken are quite unfavorable combinations of sections.

There is probably greater uncertainty as to the relative stiffness of adjoining members than there is with reference to their fixed end moments. We have already seen, however, that this uncertainty is not a very important source of error; nor is uncertainty as to the carry-over factors. The important fact seems to be that if the fixed end moments themselves can be determined it does not make so very much difference how within reason we distribute the unbalanced moment nor does it make very much difference how much of this distributed moment we carry over to the next joint. It seems probable that the true moments in those continuous girders which are not affected by continuity of members crossing them can be determined quite as accurately as in most problems in structural analysis.

Two other factors affect the moments in girders continuous with members framing into them. In the case of ordinary slab and girder construction there must be a considerable restraint at the ends of beams which frame into heavy girders because of torsional effects in the girders and some torsional restraint of the girders from their connecting beams. We know very little about the matter. There seem to be practically no published data indicating the comparative stiffness of concrete girders in bending and in torsion. But it is not difficult to imagine that twisting a concrete girder must require a moment quite comparable with the necessary to bend it, especially where the girder is continuous with a slab which practically prevents torsional distortion at the top. The effect of this torsion is evidently to make all beams more nearly fixed ended than one would at first suppose.

In addition to this torsion there is in slab and girder construction some variation in the loads brought to the girders from the beams. If two adjacent panels are completely loaded and the outer ends of the beams are free, the girder should receive about 25 per cent more load from the beams than it is ordinarily assumed to carry. The outer ends of the beams are rarely free but there will in general be some increase in the load on the girder due to the influence of the continuity of the beams and some increase in the load brought to the beams by their slabs.

The torsional rigidity of members framing in at the ends of the span tends to make each span act as an isolated fixed ended beam and therefore not subject to additional moments brought into it from other spans, while the continuity of slabs over beams or of beams over girders tends to increase somewhat the load carried by the member under consideration. In the case of large members of long span these elements are usually less important; certainly the torsional element is relatively of less importance.

These elements are pointed out here as a warning against too naive comparison between conventional coefficients and academic theoretical values for the moments. While the moments in an isolated frame work may perhaps be determined within 10 or 15 per cent other uncertainties enter into the action of typical slab and girder construction and still others into flat slab construction.

Moreover it is well to be warned against a comparison of the moments determined by the application of the conventional coefficients to the moments on the clear span with the results obtained at centers of supports by theoretical analysis. Moments at the center of intersection of the members must be reduced in order to determine the moment at the face of the column. It is in the moments at the face of the column that we are interested and we can not tabulate and compare these with the moments determined by conventional methods unless we assume a certain definite ratio of column width to span. Comparison becomes especially unsatisfactory if we undertake to compare the moments given by conventional rules for haunched beams with moments which theory would indicate.[1]

The moments at the column face may be got from moments at the column center by sketching in the curves of moment maxima as shown above.

Time yield of the concrete is often cited as an important source of uncertainty in determining moments in continuous construction, especially in the case of dead load moments. The idea seems to the writer to have little to support it. To enter accurately into the question here would lead too far afield. The idea seems to be based on the conception that a column, for example, subject to heavy moment from dead load of connecting girders will slowly yield and get rid of its moment. This seems to neglect the fact that the girders also yield, follow up the column and so continue to apply a moment to it. These terms are vague, but it may be said in passing that theoretical studies[2] show no reason to suppose that time yield seriously affects moments produced by loads. That it does affect in an important way moments produced by temperature or by settling supports is another matter.

Other sources of uncertainty, the effect of the stiffness in the joints themselves and temperature and shrinkage effects are mentioned in succeeding sections.

EFFECT OF THE JOINT

It is important to understand that the theory—any of the methods —of continuous girders assumes that rotation takes place about the center of intersection of the members. We have further assumed here, in treating members of constant section, that this constant section continues to this center of intersection.

---

[1] With whom did the conventional rules for effect of haunching originate? What is their basis?

[2] Such an investigation may be made by treating time yield as a reduction of the EI values at the yielding sections, making the reduction some function of the bending moment.

Actually, even where all axes intersect at a point, any member is very much enlarged in section after it joins another member. This is equivalent to a short and deep haunch at the end of the member. Very deep haunches, however, have little more effect than haunches of moderate depth. The effect is to increase the fixed end moment somewhat and to change somewhat the relative stiffness of the members and their carry-over factors. The effect on end moments in the girders is not commonly very great—about 5 to 10 per cent in ordinary cases. The stiffness of columns connected to deep girders is, however, very much affected, which adds further to the uncertainty of column moments.

Often the axes of the members do not meet in a point. Rotation must then take place about some compromise position. This will produce a longitudinal movement of some of the members as the joint rotates. If the far ends of the members were rigidly fixed and could not move and the eccentricity great, this would set up powerful shearing stresses in the joint. The fact seems academic, however, since unless some longitudinal movement is possible, temperature will soon tear the structure to pieces. The effect of such eccentricity in ordinary construction is probably small.

TEMPERATURE AND SHRINKAGE STRESSES IN CONTINUOUS FRAMES

This subject should be mentioned here though it does not seem wise to go into it extensively. It is evident that these stresses exist both from uniform temperature change or shrinkage and from differentials of temperature between two sides of a member and from non-uniform shrinkage.

That these stresses may in some cases reach values of several hundred pounds per square inch in the concrete seems certain. For example, a temperature differential of 40 deg. between top and bottom of a series of similar beams or between inside and outside of a tier of columns would produce a stress in the concrete of over 200 lb. per sq. in. with a corresponding stress in the steel of about 3000 lb. per sq. in. Such stresses certainly make over-refined computations of other effects seem absurd.

Computation of moments and thrusts for an assumed set of conditions is not difficult by the method here shown. Some designers will find such studies enlightening.

ACCURACY OF COLUMN MOMENTS

The writer thinks that the moments in the columns can not be very accurately determined. We can make assumptions as to moments of inertia and we can go through certain analyses but it seems clear that we will not avoid fundamental difficulties thereby.

Perhaps this can be made plain as follows: Assume that at any joint in slab and girder construction we have two girders and two columns connected. Assume further, in order to simplify the picture, that both girders and columns are of the same length and are rigidly fixed at their far ends. Now assume that an unbalanced moment rotates this joint. The problem is to determine the fiber stress resulting in the column.

It will be seen that this problem is the same as that of a composite beam made up of these four members, namely, two girders with their connecting slabs and two columns. Both slabs are presumably in tension, in fact though not from the result of this rotation alone. It is now evident that the determination of true fiber stresses in such a section is scarcely practicable. It is true that the flanges of the girders are in tension but it does not follow, in fact it is almost certainly not true, that they are ineffective.

We can conveniently think about this question by applying the formula $f = \frac{Mc}{\Sigma I}$ where c is the half depth of the column and ΣI the sum of the moments of inertia of all connecting members. If the members are not of the same length, this becomes $f = \frac{Mc}{\Sigma I \frac{L}{L_1}}$, where L is the length of any one of the members and L_1 that of the column. In both cases M is the unbalanced moment at the joint.

This conception will give the same results as computation of the moments and subsequent stress analysis provided the two processes are consistent in the values used for I. The picture here presented, however, seems to point out more clearly the uncertainties of the problem.

This method also serves to distinguish two very different cases. If the column $\frac{I}{L}$ is small with reference to the value of $\frac{I}{L}$ for the girder, as in slab and girder construction, or in continuous viaducts, enlarging the column will usually increase c more rapidly than $\Sigma I \frac{L}{L_1}$ is increased and consequently the larger the column the greater the bending stress.

The reverse is true in flat slab construction. Here $\Sigma I \frac{L}{L_1}$ increases more rapidly than does c as the column size increases and the larger the column, the smaller the bending stress. In flat slab construction, however, there is uncertainty as to the value of M, the unbalanced moment in the above formula, as well as in the value of I for the slab. The fixed end moment at the wall column when the slab is loaded is less than $\frac{1}{12}$ L times the panel load. The value of this moment will evidently depend on the torsional rigidity of the wall beam if there is one.

The whole subject of types of loading to produce maximum column stresses and of the probability of such loading is also important. There is also a question whether high tension in the steel not accompanied by high compression in the concrete is especially dangerous.

The writer doubts whether one is at present justified in placing much faith in elaborate procedures for stress computation due to combined direct stress and bending in such cases as these. But he realizes that he is questioning a theory much honored in the breach, though little in the observance.

VALUE OF MOMENT OF INERTIA

Different rules have been suggested by various writers for determining the moment of inertia of the members. Fortunately it is the relative and not the absolute values which are in question. Of course the term moment of inertia is used only in an analogous sense, simply as a measure of the relation existing between bending moment and angular deformation. It does not seem clear that measurements on the deflection of girders have anything particularly to do with the determination of the moment of inertia for computation of continuous frames.

Considerable discrepancy in relative moments of inertia does not very seriously affect our results and the rule which assumes I as proportional to bd^3 seems fairly satisfactory as regards the girders alone. In determining the relative moment of inertia of the girders and columns, however, there is considerable uncertainty because of their difference in shape. Evidently it is on the safe side as regards the moments in the columns to neglect the flange in computing the moment of inertia of the girder, but the writer is inclined to think that it is too far on the safe side.

If all girders have the same section, I/L varies as 1/L; if the girder sections are designed for shear, girders of equal depth will have I/L nearly constant, for girders of the same shape (constant ratio $\frac{b}{d}$) $\frac{I}{L}$ will vary as L and for girders of equal width I/L will vary as L^2. From such relations the designer can often estimate the relative I/L values well enough before he makes an analysis for maxima.

EXTENSIONS AND MODIFICATIONS OF ABOVE METHOD

The method indicated may be extended to cover the case of settling supports. The problem is perhaps academic unless the designer wishes to assume certain values for these settlements merely to guide his judgment. In such a case fixed end moments may be computed in the girders equal to $6EI/L^2$ times the settlement assumed. It will at once be realized that there is considerable uncertainty in regard to the value of E. These fixed end moments may now be distributed through the frame. The same method applies to temperature stresses.

The treatment of moving concentrated loads by this method would justify a separate paper. The writer has found it as convenient as any method to simply move the train of loads across the spans, determine the fixed end moments for successive positions and distribute these through the structure, thus determining the moments at various points for suc-

cessive positions of the loads and from these determining maxima. A knowledge of the general shape of the influence lines will be found a convenient guide but the writer thinks influence lines are not essential here and believes that the problem can be solved more rapidly without constructing them.

This problem occurs in buildings in the determination of moments in continuous crane runways. The problem of viaducts subject to trolley car or railway loading is of course a problem in itself but the same methods can be conveniently applied there. The technique can be systematized and simplified but it will not be elaborated here.

The method has an interesting application to open-web or Vierendeel girders, but the problem is not common.

SLABS

Perhaps it would be better not to mention slabs at all, for the prin ciples here stated contribute only slightly to the study of continuous slabs But these principles will, if judiciously applied, contribute something.

Tools available to the average engineer in thinking about slabs are the limitations imposed by statics upon the total moments, principles of symmetry and assymmetry, and mental pictures of the deflected slab as a means of judging of the variation of the moments along any given section. The last tool, though inexact, is very powerful. The writer finds the idea of distributing fixed-end moments useful in revising his mental pictures of the deflected slab when affected by continuity with other slabs or by discontinuities. It is consoling to realize that there is considerable evidence that these pictures need not be very exact and that if the total statical limitations are met, the assumed distribution of the moments need not conform precisely to results of mathematical theory.

DESIGN ASPECTS OF CONTINUITY

In discussing the effect of continuity upon the design of concrete members it is necessary to consider many questions besides the mathematical evaluation of moment shears and reactions. In the first place we should consider the probability of occurrence of those split loadings which are indicated by loading alternate spans, more particularly in that fortuitous type of loading which is necessary in order to secure absolute maxima in building frames. It has been generally recognized in structural design that the simultaneous occurrence of maximum live load over large areas is less probable than over small areas. It is usually recognized in bridge design that simultaneous occurrence of maximum live load exactly spaced but discontinuous is less probable than is continuous load. In preparing a design both factors are combined; in certain cases maxima occur only when large areas are loaded with large intensity of live load and intermediate areas are entirely unloaded with live load. The engineer is justified in taking these facts into account.

In long span construction the problem of bond becomes relatively unimportant. Probably the most convenient form of the bond relation[1] states that the maximum size of rod may be determined as

$$\phi_{Max} = 4 \frac{u}{f_s} \frac{M}{V}$$

The largest permissible size of bar is determined, then, by the ratio of the maximum moment to maximum shear. It will be found that 1¼ in. bars will not usually exceed ordinary bond requirements in spans longer than 25 ft. The moments in the longer spans would seem to be more clearly defined because of the smaller importance of column and torsional action and the action of the beam seems more definite because of the absence of uncertainty as to bond slip. In fact the ordinary theories seem to become more clear in application as the span increases.

Recent specifications have introduced the question of anchorage conspicuously into concrete design. In the longer spans anchorage will not be necessary although it is probably desirable. It is difficult to define what constitutes adequate bond or adequate anchorage in continuous girders of irregular span length. There is no such thing as "the point of inflection" in a girder; each different type of loading gives rise to a different point of inflection. If we undertake to carry bars so far that the bond in them computed by the bond formula is not excessive under any conditions of loading, we are likely to find that we are attempting to provide bond on a tension bar for a shear which would be accompanied by compression in that bar. In such a case our bond would be only about half of what we supposed it to be. In order to precisely apply the ordinary bond formula to bars in continuous girders it would be necessary to determine the maximum shear at any point coincident with tension in the bars under consideration at that point. This presents an unusually elusive problem in structural analysis; to justify its solution would require much greater faith in the bond formula than the writer of this paper has.

If we reject the bond formula and substitute the theory of anchorages we find that the point beyond which we desire to anchor is also elusive. Anchorage to the third point of the girder seems to be as readily justified as rules involving theoretical analysis of each case.

Whether one thinks that curves of maximum shear are accurately applied in determining web reinforcement depends somewhat on the views which he holds as to the action of web reinforcement.[2] Practically all tests show an intimate relation between stresses on the horizontal bars, bond on the horizontal bars and shear failure. For those loadings which give maximum shears at points removed from the support, the stress in

[1] This relation may be derived directly from the classical bond formula.

[2] To the writer it seems a clear and adequate theory of web steel to say that after the concrete has cracked on a diagonal plane, the steel prevents the beam from dropping and makes $\Sigma V = 0$ across this crack, being assisted to a variable extent by shear on the concrete still intact above the crack. Hence we should provide web steel for the maximum shear which can possibly occur at any section. Perhaps this is a case in which tests tell us what usually happens, whereas the designer's chief interest is in what may happen.

the longitudinal bars is not great because of the reduction of moment by partial loading. It seems probable that failure from web shear at points away from the support is less likely for the condition of partial loading which gives maximum web shear. The writer believes that some approximation to this maximum shear should be made but he does not believe that the facts justify very precise analysis for the maximum shear curves.

An outstanding uncertainty in design arises in connection with columns subject to bending. The writer has already indicated his doubts as to the precision with which moments are to be determined due to the deflection of connecting girders. It is to be noted that[1] the analysis of a reinforced concrete column subject to sufficient flexure to produce tension in the column is unique in structural design. This is the only case in structural analysis in which stresses due to live, temperature, wind and other loads can not be added in order to secure a total. To this is to be added the general uncertainty as to the action of concrete columns in any case and the noticeable lack of published data as to the action of concrete columns in flexure. The cubic equations which are exhibited in our literature for concrete columns in flexure seem to possess a mathematical rather than a factual background.

The writer has found that the beam formula,

$$\frac{P}{A} + \frac{M_c}{I}$$

with a proportional increase in allowable compressive stresses up to the point where tension occurs in the section and with no proportional increase beyond this point seems to agree pretty well with ordinary practice. This solution, however, is not entirely satisfactory. In the case of columns which are partly in tension due to heavy double flexure or irregular section, it is not practical to apply present methods of analysis at all; yet these cases are common enough.

The above comments on the difficulties of applying elastic analysis to actual design are not intended to be pessimistic. The writer believes that we know about all that it is necessary to know in order to secure sane economical and satisfactory designs for continuous concrete structures but he thinks a word of warning against a too naive and complacent admiration of the results of mathematical analysis may not be out of place.

ECONOMY OF CONTINUOUS CONSTRUCTION

As a general rule there is no great difference in costs between continuous and non-continuous construction. American structural literature has been much affected by two view points as to the economy of continuous construction. One group of engineers, which includes many of the older structural engineers in America, has maintained that continuous construction is necessarily uneconomical—that indetermination necessarily interferes with the efficiency of the structure. In some cases this

[1] With the exception of the deflection theory of suspension bridges.

is notably true; in a series of continuous girders, for example, the sum of the maximum reactions is considerably greater than the total load. But in most cases any theoretical self-interference does not appreciably effect economy. On the other hand some engineers have undertaken to show that continuous construction is inherently economical. This also is not true in any general sense.

Economy must be determined on its merits in each particular type of structure. In the case of steel construction continuity introduces additional costs into the erection program; in concrete it is generally simpler and more economical to build the structure continuous than otherwise. Studies of continuous steel girders show potential economies of over 15 per cent in the materials in certain cases.

In the case of concrete any theoretical economy is not so easily attained if shear requirements control concrete volume; shears are increased rather than decreased by continuity. It is true that the moments are decreased by continuity but while the area of steel required at any one section is smaller, the length of steel including anchorage is greater and no reduction in tonnage results in most cases. Saving in concrete quantities may be secured by haunching the girder and this is a familiar device. Studies made under the writer's direction indicate that haunching beyond the quarter point or to a depth greater than about the depth of the beam at the center will not promise economy.

It should be noted that any promised economy from continuity decreases in general as the relative intensity of the live load increases. The separation of the curves of maximum positive and maximum negative moment is a measure of the relative intensity of the live load. If the live load is zero these two curves become identical and coincide with the curve of dead load moments. This would be the ideal case for economy due to continuity.

STIFFNESS OF CONTINUOUS CONSTRUCTION

Continuous structures do not necessarily deflect less than those which are not continuous. It is difficult to discuss definitely "rigidity" in any structure unless we define the term more carefully than is commonly done. Sometimes rigidity is measured by the total range of deflection due to live load, though the downward deflection only is more often used. Certainly we are not especially interested in dead load deflection; we camber for that. In reinforced concrete construction it is difficult to accurately predict deflection at all because of uncertainties as to the modulus of elasticity and as to the effect of residual tension in the concrete.

It is true that a continuous girder of the same section as a series of simple spans will give less maximum downward deflection under live load than will the simple spans. If the total range of live load deflection is considered instead of the downward live load deflection alone, the deflection of the continuous span will be about the same as that of the simple span.

But continuous girders and simple girders will not be of the same section. In the case of steel construction, if the beams are designed for the same maximum fiber stress and are of the same depth throughout, the deflection of the continuous span downward will be about the same as that of the simple span and its total range of deflection will be nearly twice that of the simple span.

Frequently, continuous construction will be made more shallow than simple construction, advantage being taken of the possibility of haunching. If the depth of the continuous span at the center is much less than that of the simple span the range of deflection of the continuous span will be considerably greater than that of the simple span, even if the downward deflection alone is considered.[1]

These figures are not directly applicable to concrete because so much of the material in concrete girders is idle in flexure but they serve to show that it will not do to generalize carelessly with reference to relative rigidity.

The writer doubts whether comparison of deflections is really very significant in judging stiffness. This, however, seems to be the prevailing basis for comparison and here are the facts on this basis. Acceleration due to moving loads, in many cases, is a better basis and on this basis continuous construction shows to great advantage.

The writer knows of no general relation between stiffness and economy.

SUMMARY

The studies in continuity here presented may be briefly summarized as follows:

A sufficiently accurate approximate analysis may be quickly made for any continuous frame including the effects of haunches and columns. This should give maximum curves of moment and maximum end shears.

In short span construction where torsional elements play a large part, it is doubtful whether moments can be determined with great precision. In general the tendency of the continuity of the whole structure is to increase the loads due to the continuity of connecting members and also to make the span act as an isolated fixed ended beam because of torsion.

Determination of the longitudinal steel required is quite satisfactory. Problems of bond and of web reinforcement are, however, less clear. It is not possible to directly apply, without further investigation, the curve of maximum shears to determine bond by the bond formula nor to determine anchorage.

Stresses in columns due to continuity are quite uncertain. It is quite doubtful whether the theory of continuity in such a case will give values closely enough in accordance with the facts to justify the elaborate theories applied in the analysis of such columns.

[1] Comparative studies of deflections are most readily made from the theorem that deflection at any point of a span is proportional to the bending moment on this span due to the f/y curve as a load.

Continuous structures are not necessarily more economical than are structures which are not continuous.

If a structure is continuous, as nearly all concrete structures are, it is essential to consider this continuity in the design.

Continuous structures will not as a general rule deflect less under load than those which are not continuous, provided the continuous structures are economically designed. This rule may or may not have application in concrete construction depending upon whether there is necessarily a large amount of idle material.

W. F. WAY* (*By Letter*).—Professor Cross' paper is very interesting and valuable. The treatment of the continuous beam and of framed structures by this method of distributing the unbalanced moments throughout the system greatly simplifies the statically indeterminate problem, and as application of the method becomes more general and widespread, undoubtedly a much greater use of framed structures will result. Mr. Way.

While the approximate method of solution presented by Prof. Cross is indeed simple, to recommend its use with reinforced concrete for the *reason* that the percentage of errors involved in the approximate method is somewhat less than the percentage of errors involved in concrete's fixed beam moments and modulus of elasticity, is strange logic. If these errors compensate and neutralize one another, all is well; if they do not—if they are cumulative—all is not so well. On such a basis, yet farther and even simpler approximations involving greater errors might be warranted, since available for their justification and defense are all the uncertainties and errors involved in load assumptions. It will be much better to adhere to fairly precise mathematical analysis and separately to consider the errors involved in fixed beam moments and modulus of elasticity. Professor Cross presents an exact analysis and should have concentrated upon it.

This method is not new, however. In January, 1925, in my office H. M. Hadley made use of this method of distributing unbalanced moments in the design of the Fitting Out Pier built for the Navy Department at Bremerton, Wash., see *Engineering News-Record*, Vol. 98, p. 352 (March 3, 1927). Certain phases and aspects of the problem occur in Mr. Hadley's notes of several years earlier. In 1925 Mr. Hadley prepared a brief paper on the application of this method to the solution of continuous beam problem, which was submitted to the American Society of Civil Engineers in December, 1925. The paper was read by certain members, familiar with such problems, but finally failed of acceptance. The bridge department of the City of Seattle has used Mr. Hadley's method for several years and has found it very serviceable and satisfactory.

G. E. BEGGS (*By Letter*).—Professor Cross' remarks about the stiffness and economy of continuous construction as compared to non-continuous merit comment. His statements are well guarded, but at the same time this guarding of his negative comments about the rigidity and economy of continuous structures removes vigor and value from these remarks. A thoughtful person can reach no conclusion from his logic, Prof. Beggs.

* Henry C. McFee Contracting Co., Seattle, Wash.

while a careless thinker will be led to believe that he can do no better than design his concrete structures as an assemblage of discontinuous parts.

It is without doubt the experience of craftsmen for centuries that strength and sturdiness of construction go hand in hand with continuity of construction. Rigidity is commonly and properly measured by deflection under an applied load. You can make no continuous structure more rigid by articulating or severing any of its members. This can be demonstrated beyond any danger of disproof. Our common sense also supports this view. Those who are philosophically inclined will find support in Maxwell's theorem of reciprocal deflections, from which can be proven as a corollary the general law that no continuous frame can be made more rigid by making it partially or wholly discontinuous.

The engineer or mathematician is too often persuaded by the difficulties of mathematical analysis to articulate or sever the members of a continuous structure. Rather than admit the limitations of his mathematical tools, he is inclined to condemn continuous construction as weak and uneconomical.

Mr. Lindau. A. E. Lindau (*Chairman*)—Before I refer this discussion to Prof. Cross I would like to say that I agree with Prof. Beggs in general, in that we are inclined to build structures using discontinuous members for lack of mathematical facility, but this lack of mathematical facility is not the only reason for avoiding continuous construction. A designer is some times in doubt about the results of his mathematics in computing moments and shears of elastic structures in reinforced concrete. For example, in applying the three-moment theorem to the ordinary case of school-room construction with two long spans and a short corridor span between them, the problem of determining the negative moment over the piers, as well as the positive moment in the middle of the span, involves a computation of the moment of inertia. This factor is an uncertain computation in some types of reinforced concrete floor construction and the choice of design of floor construction may cause so large a variation in the pier moments as to leave a question in the mind of the designer as to whether the theory of continuous construction is applicable to his case. In a recent example that came before the speaker, the variation in the moment of inertia seemed to cause a variation of pier moment of as much as 50 per cent and it seems to him that cases of this kind are apt to influence the designer in favor of discontinuous construction which seems to him more definite in design. Now I will refer Professor Beggs' discussion to Professor Cross.

Prof. Cross. Hardy Cross—Professor Beggs is in error in his views as to the universal advantages of continuous construction. Since the problems of economy and rigidity are of secondary importance in the paper, this scarcely seems the place to go into a lengthy discussion of them. In most cases steel structures will cost less if discontinuous; concrete structures will sometimes cost less if discontinuous; most engineers know this. Whether continuity reduces deflections depends on the relative proportions of the structure as designed with and without continuity. Max-

well's theorem has nothing to do with the question because we are comparing the deflections of two structures having different sections and different proportions. As regards the relative strength of continuous and discontinuous construction, structures designed for the same loads and stresses have about the same strength unless our whole basis of design is all wrong.

The writer holds no brief against continuous structures; nor, on the other hand, does he hold any brief for them, for he has nothing to sell. There are cases in which continuity offers distinct advantages and other cases in which there are important objections to it. Mr. Lindau calls attention to a case in which there are obvious disadvantages from continuity; another example occurs in the continuity between the end girder and end column in a series of spans. The picture of sturdiness, strength, economy and rigidity always running hand in hand with continuity to the call of the craftsman has the merit of quaintness—and no other merit.

Mr. Way objects to the writer's logic in recommending the approximate method shown. Apparently his objection is to the logic and not to the approximation if one may judge from his justification, in the *Proceedings* of the American Society of Civil Engineers, October, 1928, of still more approximate methods. Practically all methods used in structural design are more or less approximate; some designers, however, do not know this and confuse definiteness with precision. If, however, the object of engineering studies were scholasticism rather than construction, the writer would probably agree with Mr. Way.

The writer does not agree with Mr. Way that the whole paper should have been devoted to the exact method given. The writer does not like this exact method very much; the method which he recommends for exact studies is contained in another paper which, as stated, was in the hands of another Society and which will appear in time. The writer has used and explained distribution of unbalanced fixed-end moments since 1922, and during this time has developed five or six distinct procedures for distribution, each of which seems to appeal to a different type of mind.

The great need at present seems to be not abnormal precision in computation nor academic birdsegging in methods of analysis, but a serious recognition of and consistent provision for continuity as a factor in concrete construction. The paper discusses some aspects of the problem, mentions others and omits still others. The writer cannot think of any problem in concrete upon which continuity does not have an important bearing.

Design of Reinforced Concrete Slabs

By Joseph A. Wise*

Reinforced concrete slabs possess an element of strength that is often neglected in design, resulting in wasteful practices. This element of strength is what might be termed "plate strength." If a concrete slab is analyzed as though it were composed of separate beams this element of "plate strength" is neglected. "Plate strength" can be visualized as arising from the inter-action of adjacent beam strips in the beam method of analysis of plates. Many tests and analyses have shown that reinforced concrete slabs are much stronger and stiffer than "beam-strip" analyses would indicate. Attempts have been made to alter the beam strip method of analysis empirically to take advantage of this strength, but I do not believe that any such method can be successful except for a few special cases. The obvious procedure would be to develop a distinctive method of slab analysis that would be general, and would produce slab designs that are safe and yet not wasteful. In a paper[1] presented last year such a method of analysis was given. It is not a difficult method and does not require a high degree of mathematical skill for its application. This is particularly true of "statically determinate" slabs, such as single panels simply supported at the edges or corners. For the more general case of slabs continuous over a number of panels or for the girderless floor the solution even by the "method of the elastic web" is more involved. But even in the case of continuous slabs very good approximations are possible based on an initial solution of the separate panels by the method of the elastic web. Flat slab or girderless floors are now generally designed under rules based on plate theory and this entire subject has been very ably treated by Messrs. Westergaard and Slater.[2] These will not be discussed in this paper. It will be limited to a presentation of the results of analyses for special cases. One group of these special cases consists of rectangular slabs supported on the four edges and loaded with uniformly distributed or concentrated loads. A second group contains the rectangular slab supported on four edges and subject to a load increasing uniformly from one edge to the opposite edge. Structures corresponding to the cases coming under this second group are flat slab or Ambursen dams, retaining walls and other similar structures in which slabs are subject to a

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[1] "The Calculation of Flat Plates by the Elastic Web Method." Page 408, A.C.I. *Proceedings*, 1928.

[2] H. M. Westergaard and W. A. Slater. "Moments and Stresses in Slabs." A.C.I *Proceedings*, 1921.

hydrostatic pressure. A few typical designs are also added to show clearly the application to practice.

The first case to be considered will be the rectangular slab freely supported at the four edges. A typical calculation for a uniformly loaded slab whose ratio of length to breadth is 1.5 is given and the results of similar analyses for slabs whose ratios of length to breadth are 1.0, 1.1, 1.2, 1.3, 1.4 are also given. The case of a concentrated load at the center is also given.

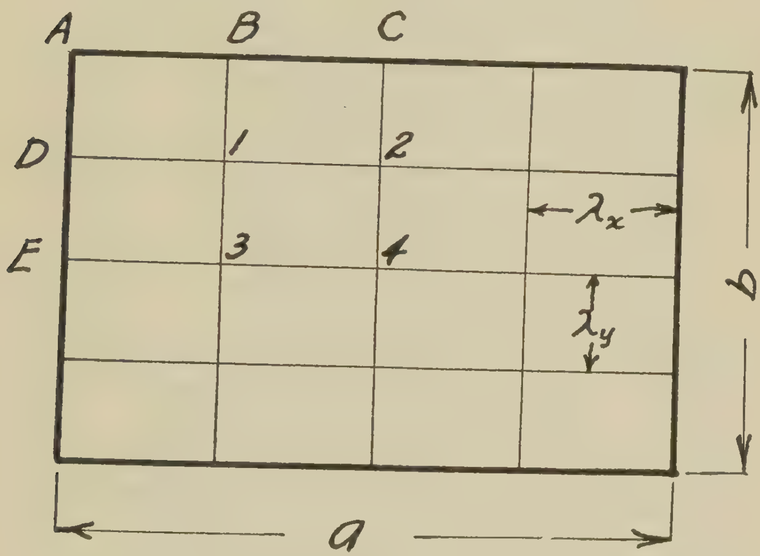

FIG. 1.

Consider first the rectangular slab freely supported at the four edges, shown in Fig. 1. For the development of the equations used see the paper previously mentioned.[1]

$$p = \text{intensity of uniformly distributed load}$$

$$\frac{a}{b} = \frac{3}{2} = k \qquad k^2 = \frac{9}{4}$$

$$\text{Then, } 2w_k\left(1 + \frac{9}{4}\right) - (w_i + w_l) - \frac{9}{4}(w_m + w_n) = \frac{p\lambda_x^2}{S_1}$$

$$\text{or } 26w_k - 4(w_i + w_l) - 9(w_m + w_n) = 4\frac{p\lambda_x^2}{S_1}$$

[1] "The Calculation of Flat Plates by the Elastic Web Method." A.C.I. *Proceedings*, 1928.

Applying to this case,

$$26w_1 - 4w_2 - 9w_3 = 4\frac{p\lambda_x^2}{S_1}$$

$$-\ 8w_1 + 26w_2 - 9w_4 = 4\frac{p\lambda_x^2}{S_1}$$

$$-18w_1 + 26w_3 - 4w_4 = 4\frac{p\lambda_x^2}{S_1}$$

$$-18w_2 - 8w_3 + 26w_4 = 4\frac{p\lambda_x^2}{S_1}$$

solving these, we get as the ordinates of the elastic web,

$$S_1w_1 = .42615p\lambda_x^2 = .02663pa^2$$

$$S_1w_2 = .52249p\lambda_x^2 = .03266pa^2$$

$$S_1w_3 = .55444p\lambda_x^2 = .03465pa^2$$

$$S_1w_4 = .68616p\lambda_x^2 = .04289pa^2$$

The equations for the elastic surface are,

$$26z_1 - 4z_2 - 9z_3 = 4w_1\frac{\lambda_x^2}{S_2} = 1.70460\frac{p\lambda_x^4}{S_1S_2}$$

$$-8z_1 + 26z_2 - 9z_4 = 4w_2\frac{\lambda_x^2}{S_2} = 2.08996\frac{p\lambda_x^2}{S_1S_2}$$

$$-18z_1 + 26z_3 - 4z_4 = 4w_3\frac{\lambda_x^2}{S_2} = 2.21776\frac{p\lambda_x^2}{S_1S_2}$$

$$-18z_2 - 8z_3 + 26z_4 = 4w_4\frac{\lambda_x^2}{S_2} = 2.74464\frac{p\lambda_x^2}{S_1S_2}$$

Solving, we get

$$\zeta_1 = \frac{S_1S_2}{N}z_1 = .0008149\frac{pa^4}{N}$$

$$\zeta_2 = \frac{S_1S_2}{N}z_2 = .0010887\frac{pa^4}{N}$$

$$\zeta_3 = \frac{S_1S_2}{N}z_3 = .0011302\frac{pa^4}{N}$$

$$\zeta_4 = \frac{S_1S_2}{N}z_4 = .0015138\frac{pa^4}{N}$$

For the determination of the stress moments, the lowest value of m is the most conservative, and for reinforced concrete m will be taken as equal to 5. Then

$$S_x = \frac{N}{\lambda_x^2}[(2\zeta_k - \zeta_i - \zeta_l) + \frac{1}{5}\cdot\frac{9}{4}(2\zeta_k - \zeta_m - \zeta_n)]$$

$$= \frac{4N}{a^2}[4(2\zeta_k - \zeta_i - \zeta_l) + \frac{9}{4}(2\zeta_k - \zeta_m - \zeta_n)]$$

$$S_y = \frac{4N}{a^2}[9(2\zeta_k - \zeta_i - \zeta_l) + \frac{4}{5}(2\zeta_k - \zeta_m - \zeta_n)]$$

$$t_{xy} = \frac{N}{a^2}\cdot\frac{24}{5}[(\zeta_p + \zeta_q) - (\zeta_o + \zeta_r)]$$

Also the shear at any point =

$$v_x = \frac{S_1}{2\lambda_x}(w_l - w_i)$$

$$v_y = \frac{S_1}{2\lambda_y}(w_n - w_m)$$

and the reactions on the supporting girder,

$$r_x = v_x + \frac{m-1}{m}\frac{S_1S_2}{\lambda_x\lambda_y^2}(2z_l - z_p - z_r)$$

$$r_y = v_y + \frac{m-1}{m}\frac{S_1S_2}{\lambda_x^2\lambda_y}(2z_m - z_o - z_p)$$

These are evaluated in Table 1.

In the same way, Tables 2, 3, 4, 5, and 6 are obtained for the slabs having ratios of $\frac{a}{b}$ = 1.0, 1.1, 1.2, 1.3, 1.4. Diagrams 1 to 6 inclusive show these results graphically.

Tables 7 to 12 inclusive and Diagrams 7 to 12 inclusive show the results for concentrated loads at the center. Tests have shown, as one would expect, that slabs bearing concentrated loads possess more strength than the analysis would indicate because the high stresses are localized. When the material in the locality of the concentrated load begins to pass the yield point, the adjacent portions of the slab take more of the load, resulting in a sort of redistributing effect. Consequently, for concentrated loads it was considered sufficiently conservative to use a value of m = infinity.

For continuous slabs, the number of possible cases are so great that any set of tables or diagrams could cover only a few of them. Professor Westergaard has given proposed moment coefficients in a paper presented

BENDING MOMENTS, TORSION, SHEAR AND REACTIONS OF SLAB IN FIG. 1

UNIFORMLY DISTRIBUTED LOAD

TABLE 1

$a/b = 3/2$

Point	s_x	s_y	t_{xy}	v	r
1	$.0123pa^2$	$.0211pa^2$	....	....	....
2	$.0135pa^2$	$.0218pa^2$	....	....	....
3	$.0165pa^2$	$.0289pa^2$	....	....	....
4	$.0188pa^2$	$.0303pa^2$	....	....	....
A	....	....	$+.0156pa^2$	....	....
B	....	....	....	$.243pa$	$.285pa$
C	....	....	$+.0073pa^2$	$.279pa$	$.329pa$
D	....	....	....	$.232pa$	$.290pa$
E	....	....	....	$.264pa$	$.337pa$

$C = .0312pa^2 = .0468pab$

TABLE 2

$a/b = 1.0$

Point	s_x	s_y	t_{xy}	v	r
1	$.0258pa^2$	$.0258pa^2$	$.0129pa^2$	....	....
2	$.0313pa^2$	$.0313pa^2$	0	....	....
3	$.0313pa^2$	$.0313pa^2$	0	....	....
4	$.0422pa^2$	$.0422pa^2$	0	....	....
A	....	....	$.0273pa^2$	....	....
B	....	....	$.0188pa^2$	$.172pa$	$.242pa$
C	....	....	0	$.219pa$	$.300pa$
D	....	....	$.0188pa^2$	$.172pa$	$.242pa$
E	....	....	0	$.219pa$	$.300pa$

$C = .0546pa^2$

TABLE 3

$a/b = 1.1$

Point	s_x	s_y	t_{xy}	v	r
1	$.0158pa^2$	$.0247pa^2$	$.00959pa^2$	....	....
2	$.0262pa^2$	$.0328pa^2$	0	....	....
3	$.0294pa^2$	$.0304pa^2$	....	....	....
4	$.0355pa^2$	$.0408pa^2$	....	....	....
A	....	....	$.0204pa^2$	....	....
B	....	....	$.0139pa^2$	$.285pa$	$.348pa$
C	....	....	0	$.330pa$	$.407pa$
D	....	....	$.0140pa^2$	$.281pa$	$.324pa$
E	....	....	0	$.324pa$	$.406pa$

$C = .0408pa^2$

TABLE 4

$a/b = 1.2$

Point	s_x	s_y	t_{xy}	v	r
1	$.0189pa^2$	$.0235pa^2$	$.00865pa^2$	....	....
2	$.0220pa^2$	$.0310pa^2$	0	....	....
3	$.0253pa^2$	$.0292pa^2$	0	....	....
4	$.0299pa^2$	$.0391pa^2$	0	....	....
A	....	....	$.0221pa^2$	0	....
B	....	....	$.0150pa^2$	$.274pa$	$.330pa$
C	....	....	0	$.323pa$	$.386pa$
D	....	....	$.0152pa^2$	$.266pa$	$.332pa$
E	....	....	0	$.307pa$	$.388pa$

$C = .0442pa^2 = .05304pab$

TABLE 5

$a/b = 1.3$

Point	s_x	s_y	t_{xy}	v	r
1	$.0162pa^2$	$.0222pa^2$	$.0092pa^2$	....	....
2	$.0187pa^2$	$.0292pa^2$	....	....	....
3	$.0218pa^2$	$.0280pa^2$	....	....	....
4	$.0253pa^2$	$.0372pa^2$	....	....	....
A	....	....	$.0197pa^2$	....	....
B	....	....	$.0133pa^2$	$.263pa$	$.314pa$
C	....	....	....	$.306pa$	$.362pa$
D	....	....	$.0136pa^2$	$.253pa$	$.316pa$
E	....	....	....	$.291pa$	$.369pa$

$C = .0394pa^2 = .0512pab$

TABLE 6

$a/b = 1.4$

Point	s_x	s_y	t_{xy}	v	r
1	$.0141pa^2$	$.0210pa^2$	$.0082pa^2$	....	....
2	$.0159pa^2$	$.0274pa^2$	....	....	....
3	$.0189pa^2$	$.0265pa^2$	....	....	....
4	$.0215pa^2$	$.0351pa^2$	....	....	....
A	....	....	$.0176pa^2$	....	....
B	....	....	$.0118pa^2$	$.253pa$	$.299pa$
C	....	....	....	$.291pa$	$.339pa$
D	....	....	$.0122pa^2$	$.242pa$	$.294pa$
E	....	....	....	$.277pa$	$.342pa$

$C = .0351pa^2$

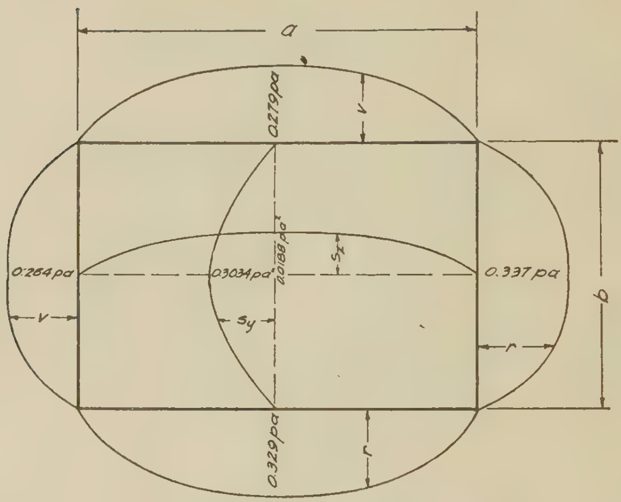

Diagram 1. $\frac{a}{b} = 1.5$

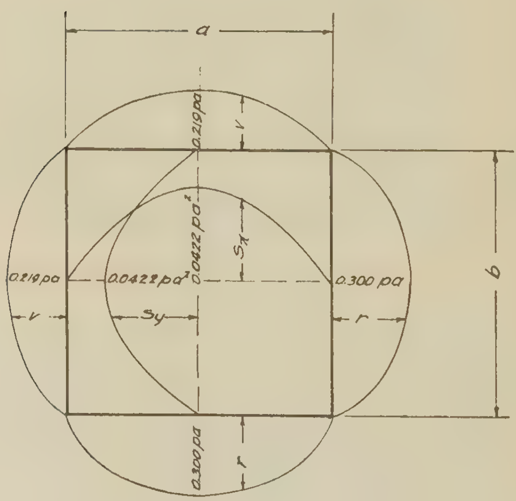

Diagram 2. $\frac{a}{b} = 1.0$

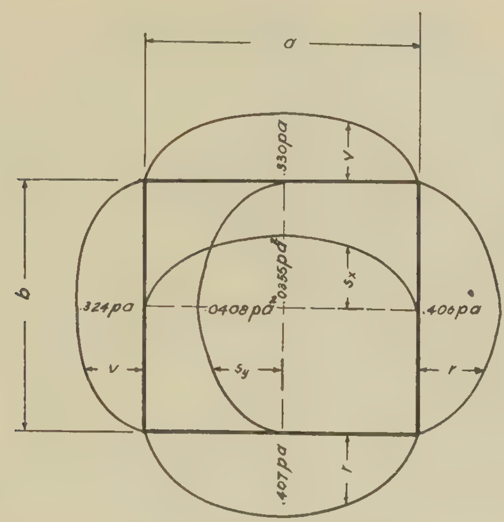

Diagram 3 $\frac{a}{b} = 1.1$

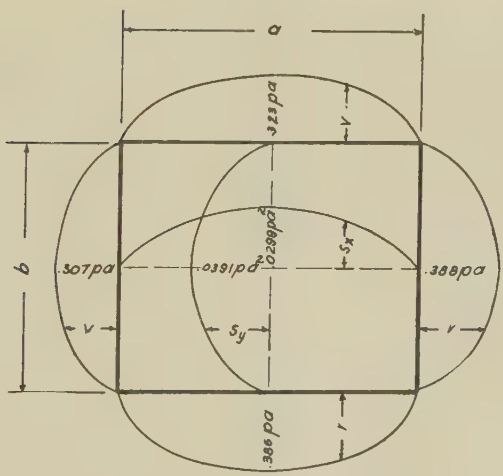

Diagram 4 $\frac{a}{b} = 1.2$

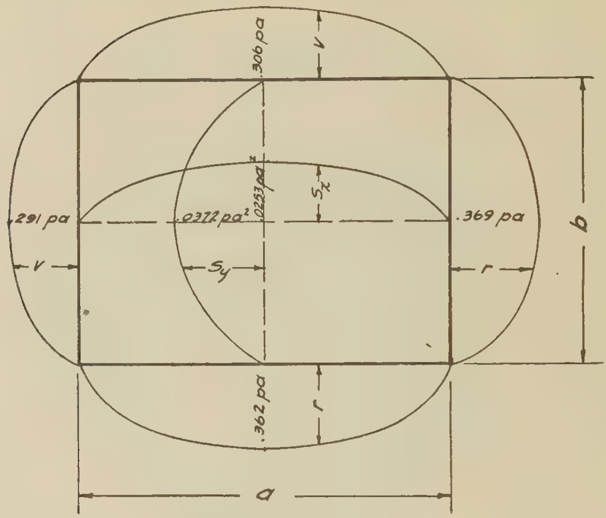

Diagram 5. $\frac{a}{b} = 1.3$

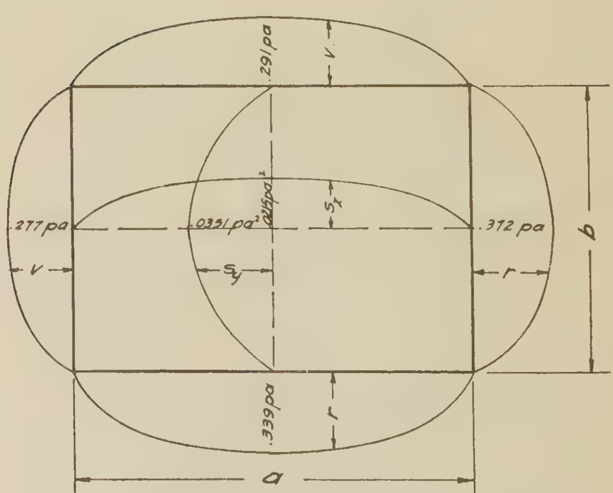

Diagram 6. $\frac{a}{b} = 1.4$

CONCENTRATED LOAD AT CENTER

TABLE 7

$a/b = 1.0$

Point	s_x	s_y	v	r
1	$.034P$	$.034P$	....	....
2	$.091P$	$.091P$	....	....
3	$.091P$	$.091P$	....	....
4	$.183P$	$.183P$	....	....
B	....	....	$.267\frac{P}{a}$	$.421\frac{P}{a}$
C	....	....	$.445\frac{P}{a}$	$.788\frac{P}{a}$
D	....	....	$.267\frac{P}{a}$	$.421\frac{P}{a}$
E	....	....	$.445\frac{P}{a}$	$.788\frac{P}{a}$

$Ca = .166P$

TABLE 8

$a/b = 1.1$

Point	s_x	s_y	v	r
1	$.027P$	$.042P$	....	....
2	$.089P$	$.057P$	....	....
3	$.037P$	$.091P$	....	....
4	$.175P$	$.190P$	....	....
B	....	....	$.290\frac{P}{a}$	$.436\frac{P}{a}$
C	....	....	$.507\frac{P}{a}$	$.877\frac{P}{a}$
D	....	....	$.253\frac{P}{a}$	$.430\frac{P}{a}$
E	....	....	$.409\frac{P}{a}$	$.757\frac{P}{a}$

$C_A = .164P$

TABLE 9

$a/b = 1.2$

Point	s_x	s_y	v	r
1	.021P	.042P	....	....
2	.085P	.049P	....	....
3	.019P	.090P	....	....
4	.169P	.196P	....	....
B	....	....	$.308\frac{P}{a}$	$.446\frac{P}{a}$
C	....	....	$.569\frac{P}{a}$	$.951\frac{P}{a}$
D	....	....	$.239\frac{P}{a}$	$.435\frac{P}{a}$
E	....	....	$.373\frac{P}{a}$	$.719\frac{P}{a}$

$C_A = .158P$

TABLE 10

$a/b = 1.3$

Point	s_x	s_y	v	r
1	.017P	.041P	....	....
2	.081P	.052P	....	....
3	.009P	.089P	....	....
4	.162P	.201P	....	....
B	....	....	$.320\frac{P}{a}$	$.455\frac{P}{a}$
C	....	....	$.626\frac{P}{a}$	$1.013\frac{P}{a}$
D	....	....	$.224\frac{P}{a}$	$.440\frac{P}{a}$
E	....	....	$.338\frac{P}{a}$	$.672\frac{P}{a}$

$C_A = .147P$

TABLE 11

$a/b = 1.4$

Point	s_x	s_y	v	r
1	$.012P$	$.038P$	....	....
2	$.077P$	$.059P$	....	....
3	$.003P$	$.088P$	....	....
4	$.156P$	$.204P$	....	....
B	....	....	$.328\frac{P}{a}$	$.461\frac{P}{a}$
C	....	....	$.685\frac{P}{a}$	$1.066\frac{P}{a}$
D	....	....	$.208\frac{P}{a}$	$.442\frac{P}{a}$
E	....	....	$.306\frac{P}{a}$	$.617\frac{P}{a}$

$C_A = .1316P$

TABLE 12

$a/b = 1.5$

Point	s_x	s_y	v	r
1	$.009P$	$.034P$	....	....
2	$.072P$	$.069P$	....	....
3	$.002P$	$.086P$	....	....
4	$.150P$	$.207P$	....	....
B	....	....	$.330\frac{P}{a}$	$.464\frac{P}{a}$
C	....	....	$.742\frac{P}{a}$	$1.107\frac{P}{a}$
D	....	....	$.191\frac{P}{a}$	$.443\frac{P}{a}$
E	....	....	$.275\frac{P}{a}$	$.555\frac{P}{a}$

$C_A = .112P$

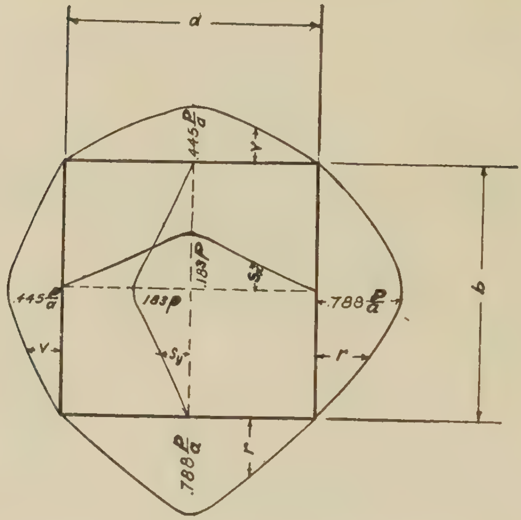

Diagram 7. $\frac{a}{b} = 1.0$

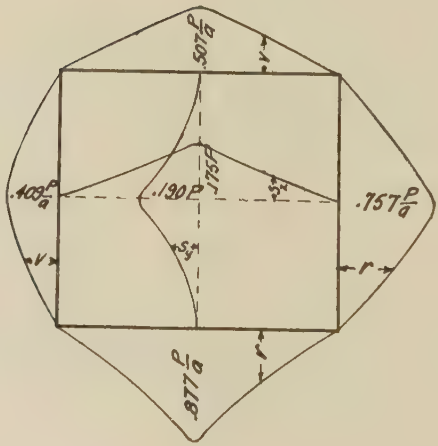

Diagram 8. $\frac{a}{b} = 1.1$

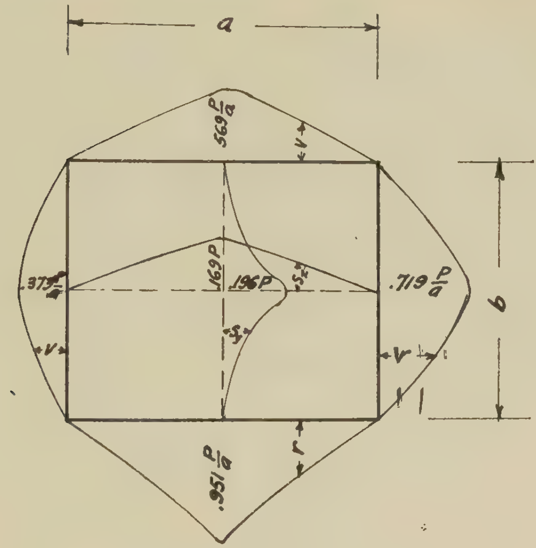

Diagram 9. $\frac{a}{b} = 1.2$

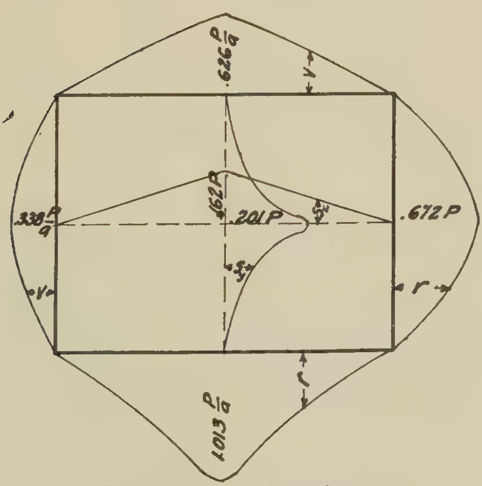

Diagram 10. $\frac{a}{b} = 1.3$

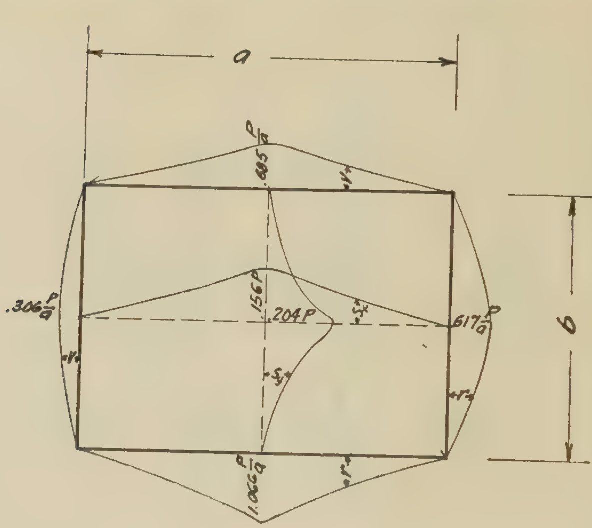

Diagram 11 $\frac{a}{b} = 1.4$

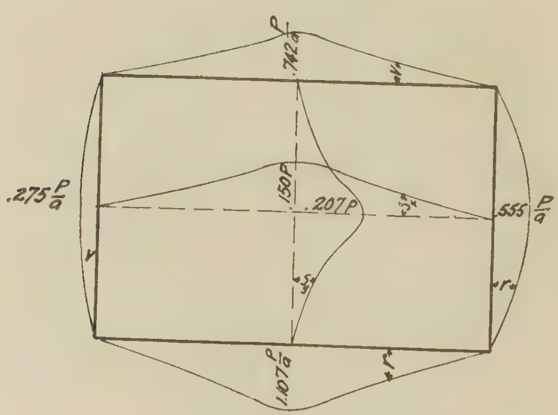

Diagram 12 $\frac{a}{b} = 1.5$

to the American Concrete Institute in 1926.[1] The values given therein are somewhat less conservative than those given in this paper. They represent average values for a width of half the panel and with m assumed equal to infinity. They probably give the moment coefficients quite closely for ultimate loads, while the ones given in this paper represent more closely the stress conditions under design loads. However, there is one question to be carefully considered concerning the continuous slab. The coefficients given by Professor Westergaard do not seem to consider sufficiently the effect of loading some panels and not loading others. This effect is most marked in the case of a single row of panels. In that case, analyses[2] have indicated that the negative moment in the slab at the center of the girder due to partial loading may be as high as $.079pa^2$; almost $\frac{1}{12}\,pa^2$. For very high ratios of live load to dead load, therefore, it would seem that

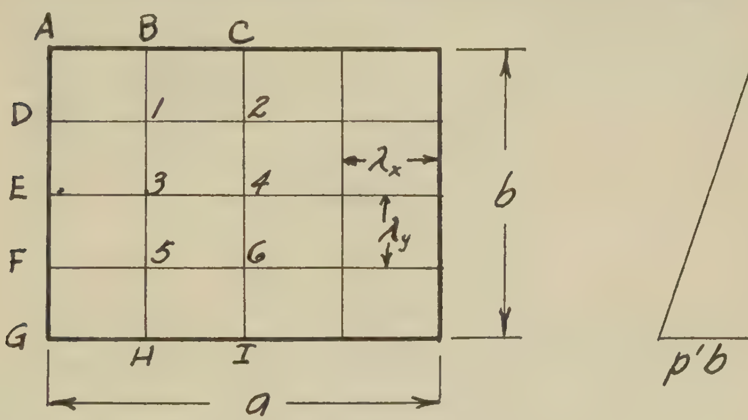

FIG. 2.

the moment coefficient of $\frac{1}{20}$ recommended by Professor Westergaard might be too low. The moment coefficients given by Professor Westergaard and those given in this paper can be combined advantageously.

Another group of cases to be considered, is that of slabs subject to load intensity increasing uniformly from zero at one edge to a maximum at the other edge. Fig. 2 indicates one type, supported on four edges.

p' = intensity of pressure at unit depth (= unit weight of liquid in cases of hydrostatic pressure).

Tables 13 to 17 inclusive and Diagrams 13 to 17 inclusive show the results for this case for ratios of $\frac{a}{b}$ = 1.0, 1.25, and 1.5 and for ratios of $\frac{b}{a}$ = 1.25 and 1.5.

[1] Formulas for the Design of Rectangular Flat Slabs and the Supporting Girders.

[2] Dr. Ing H. Marcus. "Die Theorie elasticher Gewebe und hire Anwendung auf die Berechnung biegsamer Platten." (Julius Springer, 1925.)

HYDROSTATIC PRESSURE

TABLE 13

$a/b = 1.0$

Point	s_x	s_y	v	r
1	$.0106p'a^3$	$.0058p'a^3$	....	....
2	$.0133p'a^3$	$.0115p'a^3$	....	....
3	$.0172p'a^3$	$.0156p'a^3$	....	....
4	$.0211p'a^3$	$.0211p'a^3$	....	....
5	$.0152p'a^3$	$.0173p'a^3$	....	....
6	$.0180p'a^3$	$.0229p'a^3$	....	....
B	....	....	$.0714p'a^2$	$.1012p'a^2$
C	....	....	$.0904p'a^2$	$.1269p'a^2$
D	....	....	$.0949p'a^2$	$.1160p'a^2$
E	....	....	$.1719p'a^2$	$.2125p'a^2$
F	....	....	$.2020p'a^2$	$.2497p'a^2$
H	....	....	$.2255p'a^2$	$.2644p'a^2$
I	....	....	$.2534p'a^2$	$.2981p'a^2$

$C_A = .02402p'a^3$
$C_G = .03066p'a^3$

TABLE 14

$a/b = 1.25$

Point	s_x	s_y	v	r
1	$.00588p'a^3$	$.00648p'a^3$	....	....
2	$.00702p'a^3$	$.00873p'a^3$	....	....
3	$.00939p'a^3$	$.01143p'a^3$	....	....
4	$.01164p'a^3$	$.01753p'a^3$	....	....
5	$.00811p'a^3$	$.01182p'a^3$	....	....
6	$.00919p'a^3$	$.01538p'a^3$	....	....
B	....	....	$.0565p'a^2$	$.0612p'a^2$
C	....	....	$.0707p'a^2$	$.0762p'a^2$
D	....	....	$.0662p'a^2$	$.0706p'a^2$
E	....	....	$.1194p'a^2$	$.1274p'a^2$
F	....	....	$.1414p'a^2$	$.1499p'a^2$
H	....	....	$.1580p'a^2$	$.1640p'a^2$
I	....	....	$.1774p'a^2$	$.1837p'a^2$

$C_A = .0151p'a^3$
$C_G = .0183p'a^3$

TABLE 15

$a/b = 1.5$

Point	s_x	s_y	v	r
1	$.00345p'a^3$	$.00482p'a^3$	....	....
2	$.00396p'a^3$	$.00649p'a^3$	....	....
3	$.00549p'a^3$	$.00837p'a^3$	....	....
4	$.00613p'a^3$	$.01108p'a^3$	....	....
5	$.00472p'a^3$	$.00831p'a^3$	....	....
6	$.00507p'a^3$	$.01061p'a^3$	....	....
B	....	....	$.0451p'a^2$	$.0554p'a^2$
C	....	....	$.0556p'a^2$	$.0667p'a^2$
D	....	....	$.0486p'a^2$	$.0582p'a^2$
E	....	....	$.0879p'a^2$	$.1047p'a^2$
F	....	....	$.1058p'a^2$	$.1229p'a^2$
H	....	....	$.1170p'a^2$	$.1297p'a^2$
I	....	....	$.1306p'a^2$	$.1429p'a^2$

$C_A = .00953p'a^3$
$C = .01133p'a^3$

TABLE 16

$a/b = 4/5$

Point	s_x	s_y	v	r
1	$.0178p'a^3$	$.0101p'a^3$	....	....
2	$.0229p'a^3$	$.0138p'a^3$	....	....
3	$.0294p'a^3$	$.0198p'a^3$	....	....
4	$.0390p'a^3$	$.0354p'a^3$	....	....
5	$.0252p'a^3$	$.0237p'a^3$	....	....
6	$.0330p'a^3$	$.0320p'a^3$	....	....
B	....	....	$.0867p'a^2$	$.1077p'a^2$
C	....	....	$.1101p'a^2$	$.1369p'a^2$
D	....	....	$.1322p'a^2$	$.1541p'a^2$
E	....	....	$.2422p'a^2$	$.2885p'a^2$
F	....	....	$.2867p'a^2$	$.3489p'a^2$
H	....	....	$.2821p'a^2$	$.3115p'a^2$
I	....	....	$.3097p'a^2$	$.3451p'a^2$

$C_A = .0345p'a^3$
$C_G = .0470p'a^3$

TABLE 17

$a/b = 2/3$

Point	s_x	s_y	v	r
1	$.0428p'a^3$	$.0257p'a^3$	....	....
2	$.0316p'a^3$	$.0243p'a^3$	....	....
3	$.0433p'a^3$	$.0229p'a^3$	....	....
4	$.0561p'a^3$	$.0325p'a^3$	....	....
5	$.0237p'a^3$	$.0157p'a^3$	....	....
6	$.0531p'a^3$	$.0314p'a^3$	....	....
B	....	....	$.1227p'a^2$	$.2064p'a^2$
C	....	....	$.1417p'a^2$	$.2010p'a^2$
D	....	....	$.2046p'a^2$	$.2617p'a^2$
E	....	....	$.3142p'a^2$	$.3615p'a^2$
F	....	....	$.3425p'a^2$	$.3789p'a^2$
H	....	....	$.3983p'a^2$	$.4441p'a^2$
I	....	....	$.4514p'a^2$	$.5556p'a^2$

$C_A = .05668p'a^3$
$C_G = .04892p'a^3$

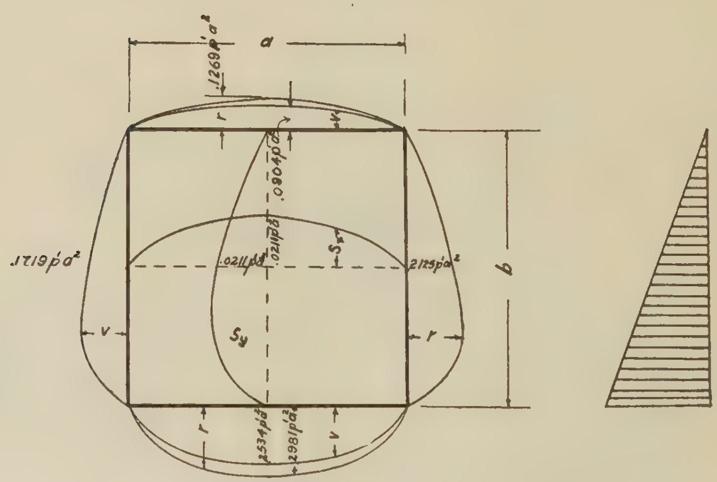

Diagram 13 $\frac{a}{b} = 1$

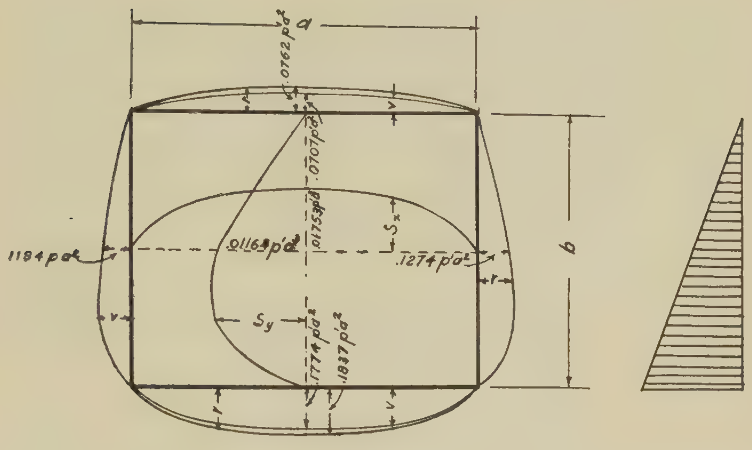

Diagram 14 $\frac{a}{b} = 1.25$

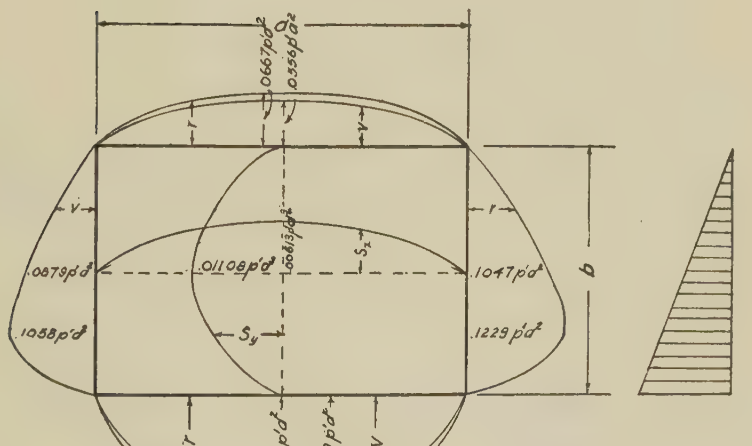

Diagram 15 $\frac{a}{b} = 1.5$

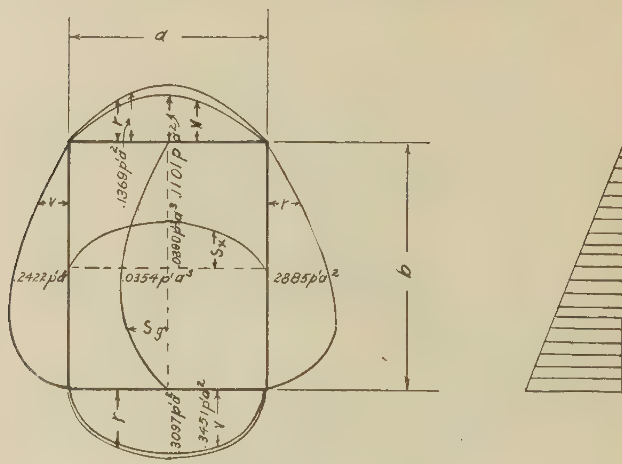

Diagram 16 $\frac{b}{a} = 1.25$

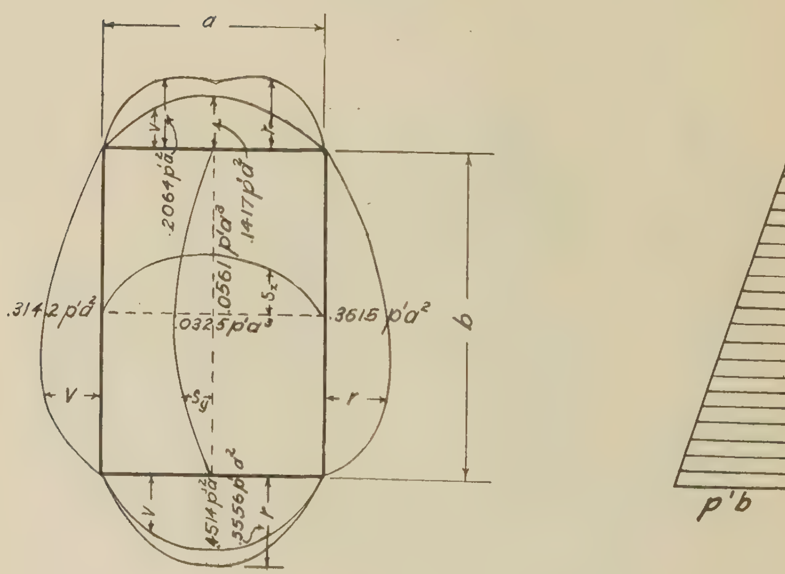

Diagram 17 $\frac{b}{a} = 1.5$

A few examples of the use of these diagrams and tables will now be given. A rectangular reinforced concrete flat slab is to be designed for a single panel, 20 × 24 ft. It is to be simply supported at the four edges. Live load is 150 lb. per sq. ft. uniformly distributed over entire panel. Allowable $f_c = 800$ lb. per sq. in., $f_s = 18000$ lb per sq. in., $n = 15$.

Assume a dead load due to weight of slab, floor finish, etc., equal to 150 lb. per sq. ft. From Table 4, the maximum moment at the center is $.0391pa^2 = .0391 \times 300 \times \overline{24}^2 = 6760$ in. lb. per in. width.

$$k = \frac{n}{n + r} = .400$$

$$j = .867,\ d^2 = \frac{6760}{\frac{1}{2} \times 800 \times .4 \times .867} = 48.75$$

$$d = 6.99 \text{ in.}$$

Use an 8½-in. slab, and ¾-in. diam. rods. Fireproofing required is ¾ in. clear. $d = 7.375$

$$A_s = \frac{6760}{18000 \times .867 \times 7.375} = .0587 \text{ sq. in. per in. width}$$

$$\text{Spacing of ¾-in. diam. rods} = \frac{.442}{.0587} = 7.53 \text{ in.}$$

Use 7½ in.

These go in the short direction. In long direction, s_y at center = $.0299pa^2 = 5170$ in. lb. These are placed above transverse rods, therefore $d = 8\frac{1}{2} - \frac{3}{4} - \frac{3}{4} - \frac{5}{16} = 6.688$ in.

$$A_s = \frac{5170}{18000 \times .867 \times 6.688} = .0495 \text{ sq. in. per in.}$$

$$\text{Spacing of ⅝ in. diam.} = \frac{.307}{.0495} = 6.2 \text{ in., use 6 in. spacing.}$$

$$p = .00765 \qquad pn = .1147$$

$$k = \sqrt{2pn + pn^2} - pn = .3776$$

$$j = .874$$

$$f_c = \frac{5170}{\frac{1}{2} \times .874 \times .378 \times (6.688)^2} = 699 \text{ lb. per sq. in.}$$

s_y at 3 is $.0292pa^2 = 5030$ in. lb.

A_s required is about .0438 sq. in. per in.

Use ¾ in. diam. at 10 in.

s_x at 2 is $.0220pa^2 = 3800$ in. lb.

A_s required is about .0361 sq. in. per in., use ⅝ in. diam. at 8½ in.

Check shearing stresses.

Maximum occurs at C, and is $.323 \times 300 \times 24 = 2325$ lb. per ft.

$$\text{Unit shearing stress} = \frac{2325}{12 \times \frac{7}{8} \times 7.375} = 30 \text{ lb. per sq. in.}$$

Bond maximum occurs at C, and unit bond stress is

$$\frac{2325}{14.73 \times \frac{7}{8} \times 7.375} = 24.5 \text{ lb. per sq. in.}$$

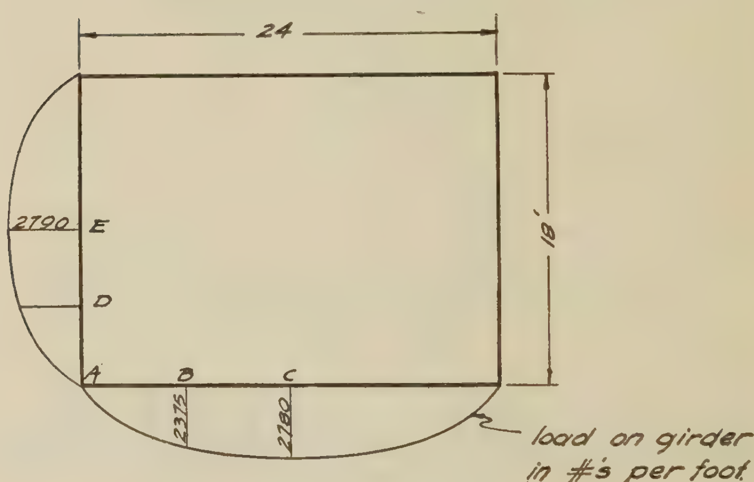

FIG. 3.

Loads on supporting girders.

These loads are shown in Fig. 3, calculated from the tabular values "r."

Lifting force in the corners.

This force is $C = .0442 \times 300 \times \overline{24}^2 = 7630$ lb.

Area of steel to provide for this is

$$\frac{7630}{18000} = .424 \text{ sq. in. or}$$

One ¾-in. diam. dowel, well anchored in the corner, is necessary.

The second example will be the design of a square flat slab dam supported on four beams.

Water pressure $p' = 62.5$ lb. per sq. ft. per ft.
Maximum moment $= .0243p'a^3$

$= .0243 \times 62.5 \times \overline{30}^3 = 4100$ ft. lb. per ft. width.
Allowable stresses; same as first example.

$$d^2 = \frac{41000 \times 12}{12 \times \frac{1}{2} \times 800 \times .4 \times .867} = 295.5$$

$d = 17.2$ in.

If a 1½-in. protective covering is required, a slab thickness of 20 in. would be necessary. For 1¼-in. sq. rods, $d = 17.875$ in.

$$A_s = \frac{41,000}{18000 \times .867 \times 17.875} = .1472 \text{ sq. in. per in. in length}$$

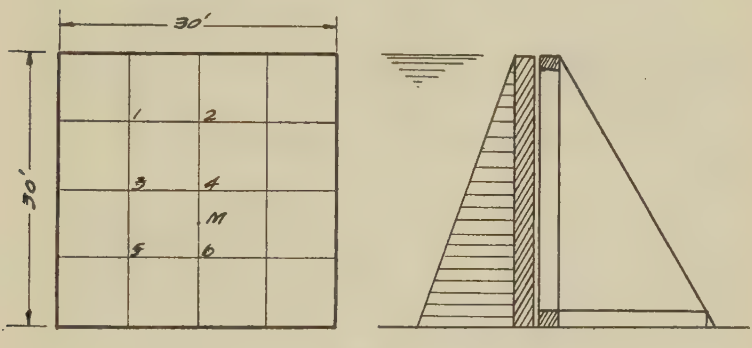

FIG. 4.

A spacing of 10½ in. is necessary for 1¼-in. sq. rods.
If horizontal steel is placed back of vertical steel, $d = 17.125$ in.
Maximum moment $= .0223 \times 62.5 \times \overline{30}^3 = 37620$ ft. lb.

$$A_s = \frac{37620}{18000 \times \frac{7}{8} \times 17.125} = .1395 \text{ sq. in. per in.}$$

or a spacing of 11-in. for 1¼-in. diam. bars.

$$p = \frac{.1420}{17.125} = .00829 \quad pn = .1243$$

$$k = .3896 \quad j = .8701$$

$$f_c = \frac{37600}{\frac{1}{2} \times .8701 \times .3896 \times (17.125)^2} = 757 \text{ lb. per sq. in. which is}$$

satisfactory.

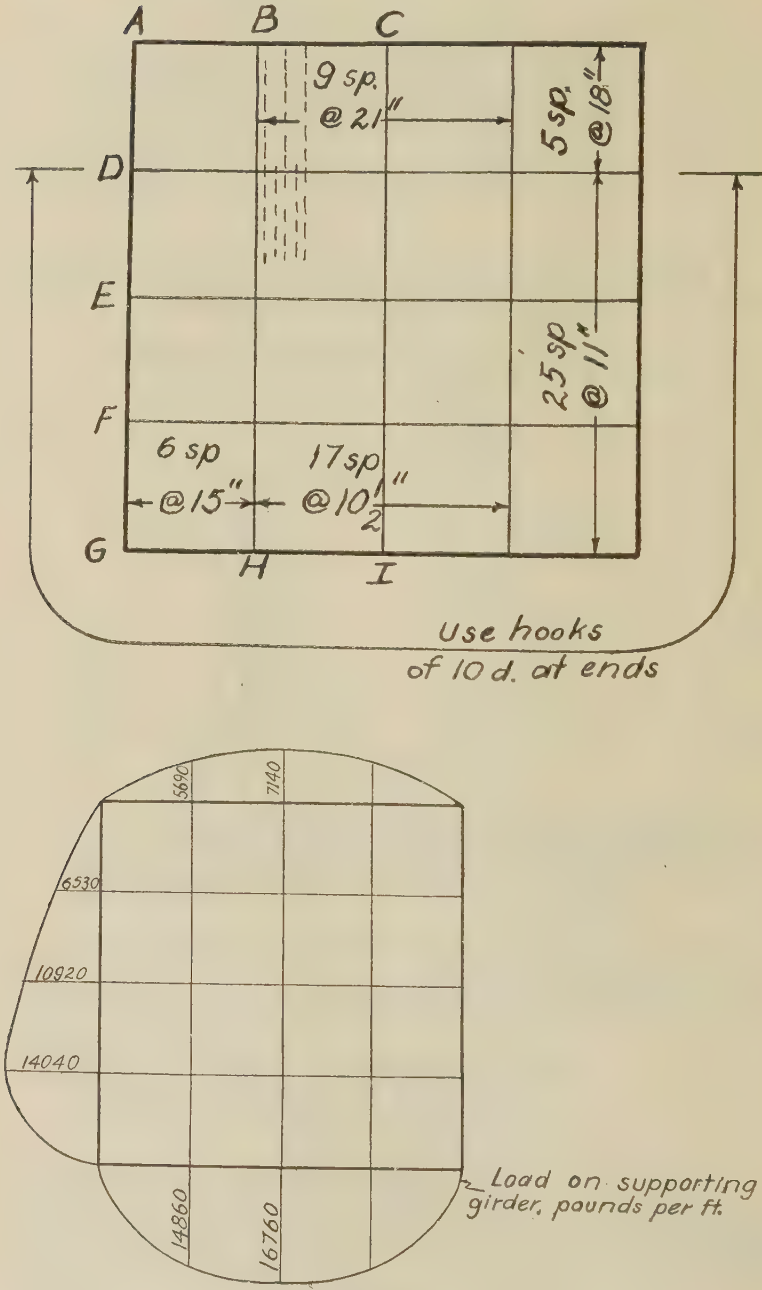
A
B
C
D
E
F
G
H
I
9 sp. @ 21"
5 sp. @ 18"
25 sp @ 11"
6 sp @ 15"
17 sp @ 10½"
Use hooks of 10 d. at ends
5690
7140
6530
10920
14040
14860
16760
Load on supporting girder, pounds per ft.

FIG. 6.

Vertical steel at 5.

$M = 29170$ in. lbs. $A_s = 1.05$ sq. in.

or space 1¼ in. sq. at 15 in.

Vertical steel at 2.

$M = 19400$ in. lbs. $A_s = .700$ sq. in.

or use 1¼-in. sq. at 21 in.

Horizontal steel at 2.

$M = 22400$ in. lbs. $A_s = .833$ sq. in.

or use 1¼-in sq. at 18 in.

Check for shear.

Maximum occurs at I, and is

$$.2534 \times 62.5 \times \overline{30}^2 = 14250 \text{ lb. per ft.}$$

Unit shearing stress

$$= \frac{14250}{12 \times \frac{7}{8} \times 17.125} \doteq 79.3 \text{ lb. per sq. in.}$$

I would not consider this value as indicating any necessity for web reinforcing.

$$\text{Bond stress; } \Sigma o = \frac{5}{10.5} \times 12 = 5.72 \text{ in. per ft.}$$

$$u = \frac{14250}{5.70 \times \frac{7}{8} \times 17.125} = 167 \text{ lb. per sq. in.}$$

Therefore, special anchorage in the form of hook ends, bent to not less than 10 diameters, will be used along the bottom and sides as far as D_1.

$$\text{At } D, \Sigma o = \frac{5}{18} \times 12 = 3.33 \text{ in. per ft.}$$

$$V = 5340 \text{ lb. per ft.}$$

$$u = \frac{5340}{3.33 \times \frac{7}{8} \times 16.625} = 110 \text{ lb. per sq. in.}$$

Therefore the hooked ends will be discontinued just above D. Final design.

From the values of r, Table 13, we get the following load curves for design of supporting girders.

The lifting force in corner G is $.03066 \times 62.5 \times \overline{30}^3 = 51{,}700$ lb., and provision must be made to resist it. In this case, $\frac{51{,}700}{18{,}000} = 2.87$ sq. in. of steel will be required. Four 1-in. diam. bars will be used as dowels and will have hooks of 10d. at each end. They will be placed, 2 on each side of the corner and will be imbedded in the slab and in the girder as deeply as possible.

Mr. Westergaard.

H. M. WESTERGAARD* (*By Letter*)—The approximate method of difference equations, whereby a differential equation is replaced by a set of simultaneous algebraic equations, is a useful tool for structural analysis. N. J. Nielsen, in his dissertation submitted at the College of Engineering in Copenhagen in 1918 (published in 1920 under the title "Bestemmelse af Spændinger i Plader ved Anvendelse af Differensligninger"), and H. Marcus, in a series of articles published in *Armierter Beton*, 1919 (and in his book of 1924) applied this method to slabs. Professor Wise deserves credit for having given publicity to the work of Marcus, and for having shown the application of the method to some cases which are of interest.

Important in connection with this method is the question of accuracy. N. J. Nielsen (p. 133 in his book) experimented on the accuracy obtained, by analyzing a uniformly loaded square slab, simply supported on four sides, using in one case a mesh consisting of 100 squares, and in another a more open mesh with only 25 squares. He found a fairly good agreement between the results obtained in the two cases, and between these results and those obtained by others by the more accurate methods, involving a direct use of the differential equation of flexure. H. Marcus performed a similar experiment with 64 and 16 squares in the mesh, respectively, and he too found the results satisfactory so far as the bending moments are concerned. A. Nádai, in his excellent book on slabs (*Die elastischen Platten*, 1925, p. 220), in the chapter dealing with difference equations, pointed out, however, that a closer mesh is required, generally, when the bending moments vary relatively rapidly, as, for example, in the case of a slab with fixed edges. The bending moments vary particularly rapidly in the immediate neighborhood of a concentrated force. By repeating the computation by means of the difference equations with the mesh closer and closer, one finds at the point of a concentrated load indefinitely increasing values of the maximum moments. Each one of these values, therefore, has little meaning. The method does not give information about the stresses produced in the immediate neighborhood of a concentrated load. This criticism applies to the values given for the center (point 4) in Tables 7 to 12 and in Diagrams 7 to 12 in the paper.

The question of concentrated loads is difficult because the usual assumption, that a straight vertical line drawn through the slab remains straight, does not apply in the immediate neighborhood of a concen-

* Professor of Theoretical and Applied Mechanics, University of Illinois.

trated load. Consequently, the usual differential equation of flexure, Lagrange's equation, also ceases to apply in this neighborhood. Nádai has given an exact solution for the case of a circular slab with a concentrated load at the center (in the book referred to, p. 315).

The phrase "stress moment" used by Professor Wise does not seem to be a fortunate translation of the German "Spannungsmoment." "Internal moment" would be better, but in most cases it is sufficient to say "moment." The important distinction is between bending moments and twisting moments, both of them internal moments, defined in terms of the stresses.

Professor Wise, in referring to the formulas proposed by the writer (A.C.I. *Proceedings*, 1926, p. 26), for design of rectangular slabs supported on four sides, intimated that some of these formulas gave too small values because of failure to consider sufficiently the effect of unloaded neighboring panels. If the coefficients are too small, it is not for this reason. The coefficients were obtained by a process described in the paper referred to. Consideration was given to the possibility of completely unloaded panels, but the phenomenon of redistribution of stresses and other reasons for reducing the coefficients were also taken into account. The purpose of the formulas is not to compute the true stresses, but to compute nominal stresses, which by comparison with the adopted working stresses lead to a judgment of the safety of the structure.

Prof. Wise.

Joseph A. Wise—Professor Westergaard's criticism of the application of the Method of the Elastic Web to plates carrying concentrated loads is sound and requires further explanation by me. Tables and Diagrams 7 to 12, inclusive, were obtained on the assumption of a load uniformly distributed over a rectangle similar to and one sixty-fourth the area of the plate. It was intended for such cases as for example a heavy safe at the center of a floor. It is impossible to apply a truly concentrated load to a slab, the load must always be applied to an area, and in most cases the area is fairly large. The tables should not be considered as applicable to forces concentrated on an area whose least dimension is less than two or three times the thickness of the plate. However, even with this reservation, it is considered that the tables will give a fair approximation to most cases occurring in practice and will enable some evaluation to be made of the effect of a so-called concentrated load. In my paper in the 1928 *Proceedings*, I discussed this problem briefly. Due to the meager experimental data on the subject, the analytical results should be regarded as tentative conclusions awaiting experimental verification.

The phrase "stress moments" was intended to apply to those moments in the slab which immediately produce tensile or compressive fiber stresses, to distinguish them from the torsional moments whose immediate effects are to produce shearing stresses. "Moment" does not seem to suffice and "internal moment" might easily be confused with the torsional moments which are also internal. It is desirable that some term be agreed upon to denote these moments and the term "flexural moment" is suggested as more appropriate than any heretofore proposed.

The question of moments in slabs continuous over beams and girders is one that requires more experimental data to definitely establish the extent to which the phenomenon of redistribution can be depended upon to reduce stresses where the analysis on the assumption of perfect elasticity indicates high values. For the present I am willing to defer to Professor Westergaard on the values that should be used.

The Absolute Basis of Proportioning Concrete and Its Economy

By Joseph A. Kitts*

Aggregates from the same source vary widely from day to day in average diameter of particles, gradation of sizes of particles, moisture content, absorption, density, specific gravity, and bulking by loose measurement and moisture. The absolute[1] volumes and gradation of sizes of particles in a mix, the water required for workability and the cement required for the strength, or other properties of the concrete, are changed by these variations of the physical characteristics of the aggregates. Fixed volumetric or weight proportions of mix may have excellent workability and quality producing characteristics one day and quite the contrary the next because of these variations in the physical characteristics of the aggregates. The variations in particular are as follows:

(*a*) Loose-moist measured volumes are affected by "bulking" or increase over the standard dry-rodded volumes. This bulking is due to looseness of measure, moisture content, and diameter and specific gravity of particles, and varies from about 0 to 40 per cent. The density varies from 50 to 75 per cent. The absolute volume of particles in a given volumetric measure varies inversely with this bulking and directly with the density.

(*b*) Inundated volumes bulk from 0 to 8 per cent depending upon the specific gravity, porosity and average diameter of particles. The amount of water required for inundation varies from 15 to 50 per cent, indicating the voids in and density of the material in the measure. The absolute volumes vary inversely with this bulking and directly with the density.

(*c*) The standard dry-rodded volume varies in density from about 50 to 90 per cent depending upon the shape, maximum size and gradation of sizes of particles, and the absolute volume in a given measure varies directly with this density. Dry-rodding is practically and scientifically useless.

(*d*) The absolute volumes, corresponding to particular weight measurements, vary inversely with the specific gravities and moisture contents of the aggregates.

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[1] "Absolute volume," as used herein, denotes "apparent volume," as generally termed by concrete physicists, and is the volume within the surface of the particles.

The effects of these variations are sometimes accumulative in one direction or the other and the strength, density, impermeability and durability, of the resulting concrete, vary widely likewise. The compressive strength of the usual loose-measured 1:2:4 mix may vary as much as 2000 lb. per sq. in. because of these changes in the materials.

It is obvious from the foregoing that arbitrary proportions, by any of the given methods of measurement, are not definite as an exact basis of mixture for uniform quality of concrete. It can also be seen that an *absolute basis of proportioning concrete mixtures must comprehend the absolute volume of aggregate particles.* However, one absolute volume may contain particles all one size and others two, three or an infinite number of sizes. To be alike, practically, they must have nearly the same gradation of sizes. The most simple equation for grading of sizes is that of Messrs. Talbot and Richart,[1] $r = 1-(d/D)^n$, in which r is the proportion retained on screen opening of d in., D is the maximum size of aggregate and n is an exponent. This equation is the basis of the tests shown graphically in Fig. 1. When $n = 0.5$, the curve is a parabola and corresponds to Fuller's theoretical grading.

It is found that strength, density, uniformity, impermeability, workability and economy of a concrete mixture depend largely upon the maximum size, coarseness and uniformity of grading of diameters of aggregate particles from fine to coarse; also, that silt (or other inert particles finer than the average cement particle) adulterates the cement and reduces the strength and density of the concrete. These requirements of maximum and minimum size, coarseness and uniformity of grading of aggregate particles are expressed by the Kitts-Peugh grading equation

$$r = \frac{1-(d/D)^m}{1-(A/D)^m}$$

in which r is the proportion by absolute volume retained on given sieve opening of d in., D is the maximum size of the aggregate, m is an exponent and A is the minimum size of the aggregate, excluding silt. We have, then, absolute volumes of particles, of any size limitations, uniformly proportioned to a definite coarseness of gradation of particles. To this we must add cement and water.

The strength, density, impermeability and economy of concrete depends also upon the absolute volume of cement to unit volume of concrete and upon the absolute volume of mixing water to unit volume of cement.

Absolute Basis of Mix

The absolute basis of a concrete mix, then, consists of:

(1) Aggregate particles having a maximum and minimum size limitation, uniformly graded to a particular coarseness by diameters and absolute volumes of particles;

[1] Bulletin No. 137, Engineering Experiment Station, University of Illinois, Urbana.

(2) An absolute volume of cement to a unit volume of concrete; and

(3) An absolute volume of mixing water to a unit volume of cement.

Example:

Having given materials of standard quality, the aggregates being rounded particles, and the requirements being a compressive strength of 3,000 lb. per sq. in. at 28 days; the mix may be specified as follows:

Strength: 3,000 lb. per sq. in. at age of 28 days.

Grading: $r = \dfrac{1-(d/D)^m}{1-(A/D)^m}$ in which

$$m = 0.5$$
$$D = 1.5 \text{ in.}$$
$$A = 0.00146 \text{ in.}$$

Cement: Absolute volume per cu. yd. of concrete = 3.395 cu. ft. = 1.75 bbl. (For density of cement 0.485.)

Water: Water to cement ratio by absolute volume = 1.85 = 0.9 cu. ft. per sack of cement.

Consistency: 6–7-in. slump.

Method of Application:

Assume that we have on the job 2 sands and 2 gravels whose characteristics at a particular time are as shown in Table 1:

TABLE 1—CHARACTERISTICS OF JOB AGGREGATES

(See Tests of Aggregates)

Practical Size Limits	Coarseness Modulus	Loose-Moist Density	Specific Gravity	Moisture by Apparent Volume	Absorption by Apparent Volume
A to No. 16	3.84	0.511	2.57	0.174	0.040
No. 100 to No. 4	5.35	0.585	2.62	0.182	0.026
No. 4 to ¾ in.	8.55	0.585	2.66	0.113	0.020
¾ in. to 1½ in.	9.83	0.581	2.65	0.061	0.020

The theoretical proportions of the mixed aggregates and of sand retained on the standard screens are determined from the grading equation $r = \dfrac{1-(d/1.5)^{0.5}}{1-(.00146/1.5)^{0.5}}$ to be as shown in Table 2.

TABLE 2—THEORETICAL GRADINGS OF MIXED AGGREGATES AND SAND FOR $D = 1\frac{1}{2}$ IN., $m = 0.5$, AND $A = 0.00146$

Sieve No.	(*d*) Inches	Proportions Retained	
		Mixed Aggregates (r)	Sand $(r-r')/(1-r')$*
A	0.00146	1.00	1.00
200	0.0029	0.99	0.97
100	0.0059	0.97	0.91
50	0.0117	0.94	0.82
30	0.0234	0.90	0.70
16	0.0469	0.85	0.54
8	0.0937	0.77	0.30
4	0.187	r'=0.67	0.00
⅜ in	0.375	0.52	....
¾ in	0.75	0.30	....
1½ in	1.5	0.00	....
Coarseness modulus	....	7.91	5.24

* r' is the proportion of coarse aggregate.

The summation of the proportions retained on each screen is a function of the average diameter of particles and is called the "coarseness modulus," the larger the value, the coarser the material. Employing the coarseness moduli we are able to determine by an algebraic process the proportions of two, three or several sizes of aggregates so that their combination most nearly conforms in grading to the theoretical grading curve, the greater the number of sizes of aggregates (A–No. 100, No. 100–No. 4, No. 4–¾-in., ¾-in.–1½-in., etc.) the more nearly we can approximate the theoretical grading by its use.

The theoretical coarseness modulus of the coarse aggregate is determined by proportion from the values in Table 2 to be:

$$\frac{7.91-5.24\ (1-r')}{r'} = 9.23 = \frac{\text{theo. total}-\text{theo. fine (proportion of fine)}}{\text{proportion of coarse}}$$

The theoretical grading of the mixed aggregate, Table 2, is separated at the No. 4 sieve corresponding to the pairs of job aggregates, as given in Table 1, and the theoretical coarseness moduli of the parts and the whole are:

COMBINATIONS	THEORETICAL COARSENESS MODULI
Sand Combination	5.24
Gravel Combination	9.23
Whole Combsnation	7.91

We have then by simple proportion

$$100\ \frac{(9.23-7.91)}{(9.23-5.24)} = 33 \text{ per cent sand} = \frac{\text{theo. coarse}-\text{theo. total}}{\text{theo. coarse}-\text{theo. fine}}$$

$$100-33 \qquad = 67 \text{ per cent gravel}$$

which check the determinations in Table 2.

In the same manner the proportions of the job aggregates are determined, from these theoretical moduli and the actual moduli given in Table 2, as follows:

$$33\ \frac{(5.35-5.24)}{(5.35-3.84)} = 3 \text{ per cent fine sand} = \frac{\text{coarse–theo. total}}{\text{coarse–fine}}$$

$$33-3 = 30 \text{ per cent coarse sand}$$

$$67\ \frac{(9.83-9.23)}{(9.83-8.55)} = 31 \text{ per cent fine gravel}$$

$$67-31 = 36 \text{ per cent coarse gravel}$$

Total.... 100 per cent

These are the proportions of dry aggregate by absolute volume while the given coarseness moduli are prevailing; they are the practical proportions, for the given coarseness moduli, most nearly fitting the theoretical grading.

The absolute volume of mixed aggregate in 1 cu. yd. of concrete of the given specifications will then be

$$(27/\text{yield}) - \underset{\text{CEMENT}}{3.395} - \underset{\text{WATER}}{(3.395 \times 1.85)} = 17.324 \text{ cu. ft./yield}$$

Assuming the "Yield" as unity, we have then

		Absolute Volume	
Cement.....	$1.75 \times 4 \times 0.485$	= 3.395 cu. ft.	= bbl. × 4 × density
Mixing water	1.85×3.395	= 6.281 "	= ratio × abs. vol. cement
Fine sand...	17.324×0.03	= 0.520 "	= abs. vol. mixed aggregate × proportion
Coarse sand.	17.324×0.30	= 5.197 "	
Fine gravel..	17.324×0.31	= 5.370 "	
Coarse gravel	17.324×0.36	= 6.237 "	
Total......................		27.000 "	

The correction for moisture content and absorption of the individual aggregates is 6.281–0.52 (0.174–0.04)–5.197 (0.182–0.026)–5.37 (0.113-0.02)–6.237 (0.061–0.02) = 4.645 cu. ft. = 34.8 gal. to be added.

The proportions, measuring aggregates by loose-moist volume, are:

		Loose Volume	
Cement...............	1.75×4	= 7 sacks	= bbl. × 4
Added water...........	....	34.8 gal.	= as corrected above
Fine sand.............	0.520/0.511	= 1.0 cu. ft.	= $\frac{\text{abs. vol. of each agg.}}{\text{loose moist density}}$
Coarse sand...........	5.197/0.585	= 8.9 "	
Fine gravel............	5.370/0.585	= 9.2 "	
Coarse gravel..........	6.237/0.581	= 10.7 "	

The proportions by moist weight are:

		Moist Weight	
Cement......	$1.75 \times 4 \times 94$	= 658 lb.	= 376 lb. per bbl.
Added water..	4.645×62.4	= 289 "	= cu. ft. × 62.4
Fine sand....	$0.520 \times (2.57 + 0.174) \times 62.4$	= 89 "	= abs. vol. × (sp. gr. + moisture) × 62.4
Coarse sand..	$5.197 \times (2.62 + 0.182) \times 62.4$	= 909 "	
Fine gravel...	$5.370 \times (2.66 + 0.113) \times 62.4$	= 929 "	
Coarse gravel.	$6.237 \times (2.65 + 0.061) \times 62.4$	= 1055 "	
Total..................................		3929 "	

This is merely simple physics and arithmetic.

FIG. 1—COMPOSITE DIAGRAMS OF THE CHARACTERISTICS OF BASIC CONCRETE MIXTURES OF VARIOUS CEMENT AND WATER CONTENTS AND GRADINGS OF AGGREGATES.

The same coordinates on the two ("Economic" and "Laboratory") charts represent the same basic concrete mixture. Thus, any point on the chart indicates a basic mix and the values of the characteristics of the mixture are shown as contour lines around that point. The basic mixture is determined by: (1) the basic grading of aggregate, $r = 1 - (d/D)^n$; (2) the cement content, bbl. per cu. yd. of concrete; and (3) the water to cement ratio, w/c.

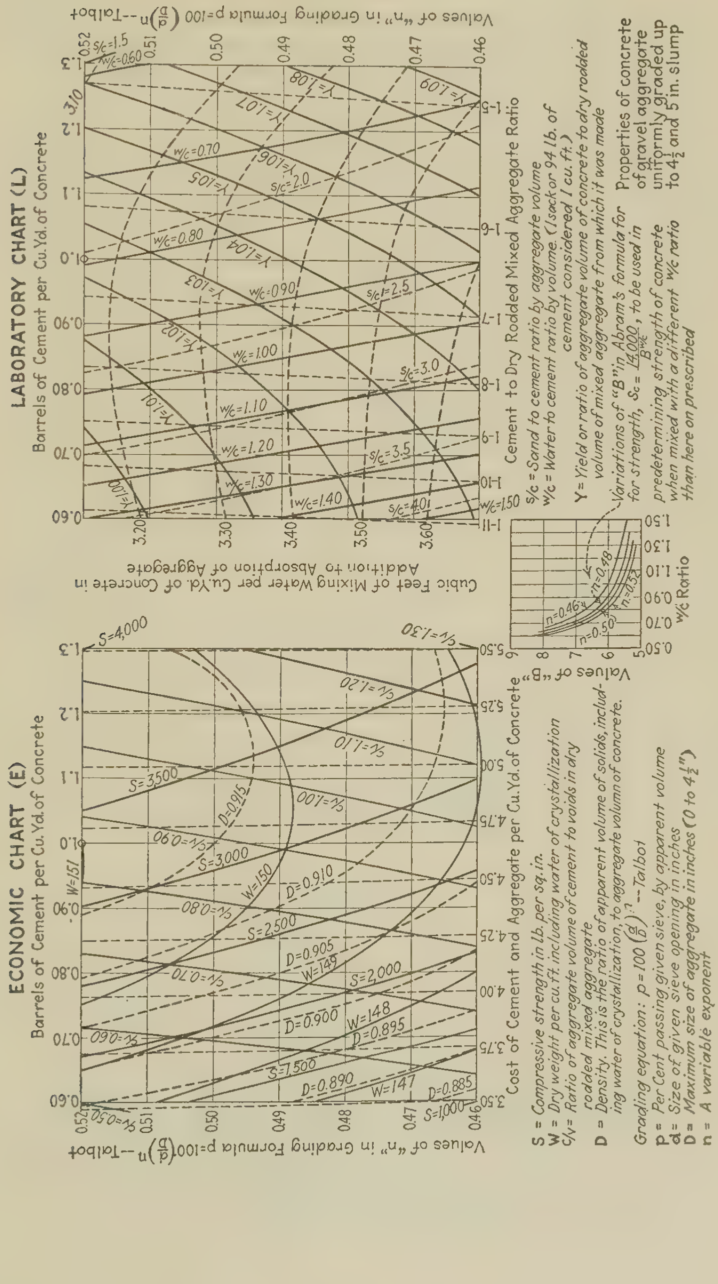
Example:

COORDINATES OF A BASIC MIX:		SHOWN ON CHART
(1) $n = .52$ in. $(r = 1 - (d/4.5)^{.52})$		E & L
(2) bbl. of cement per cu. yd. = 1.0		E & L
(3) w/c = 0.795		L
OTHER CHARACTERISTICS OF SAME MIX:		
Weight per cu. ft.	= 151.0 lb.	E
Strength at 28 days	= 3300 lb. per sq. in.	E
Density	= 0.92	E
Cost per cu. yd.	= $4.70	E
Cement to voids in aggregate	= 0.855	L
Yield of concrete	= 1.024	L
Sand to cement	= 2.02	L
Mixing water per cu. yd.	= 3.14 cu. ft.	L
"True mix"	= 1:6.6	L

Economy factor = 3300/$4.70 = 700 lb. strength per dollar cost of materials.

Compressive strength = 5830 -- 1250 S/C = $9000/3.7^{w/c}$ approximately.

(Acknowledgment is made to Messrs. Lewis H. Tuthill and Verne L. Peugh who, in 1926, designed this method of graphical analysis of the physics of concrete mixtures.)


LABORATORY CHART (L)
Barrels of Cement per Cu. Yd. of Concrete
Values of "n" in Grading Formula p=100 (d/D)n--Talbot
Cubic Feet of Mixing Water per Cu. Yd. of Concrete in Addition to Absorption of Aggregate
Cement to Dry Rodded Mixed Aggregate Ratio
s/c = Sand to cement ratio by aggregate volume
w/c = Water to cement ratio by volume. (1 sack or 94 lb. of cement considered 1 cu. ft.)
Y = Yield or ratio of aggregate volume of concrete to dry rodded volume of mixed aggregate from which it was made
Variations of "B" in Abram's formula for strength, Sc = 14,000/B^w/c, to be used in predetermining strength of concrete when mixed with a different w/c ratio than here on prescribed
Properties of concrete of gravel aggregate uniformly graded up to 4½ and 5 in. slump
Values of "B"
w/c Ratio
ECONOMIC CHART (E)
Barrels of Cement per Cu. Yd. of Concrete
Cost of Cement and Aggregate per Cu. Yd. of Concrete
Values of "n" in Grading Formula p=100(d/D)n--Talbot
S = Compressive strength in lb. per sq. in.
W = Dry weight per cu. ft. including water of crystallization
c/v = Ratio of aggregate volume of cement to voids in dry rodded mixed aggregate
D = Density. This is the ratio of apparent volume of solids, including water of crystallization, to aggregate volumn of concrete.
Grading equation: p = 100 (d/D)^n --Talbot
p = Per Cent passing given sieve, by apparent volume
d = Size of given sieve opening in inches
D = Maximum size of aggregate in inches (0 to 4½")
n = A variable exponent

Constant Control Required—It should be kept in mind that these calculated proportions are correct only as long as the characteristics of the aggregates remain as given in Table 1 and that 1 man-hr. of systematized and coordinated laboratory control is required for every 10 to 50 cu. yd., depending upon the conditions, in order to keep up with the change of materials with the most economical results. Preliminary tests should be made, with the materials to be used, varying the grading of aggregate and cement content—the weight, density, yield, strength, and the ratios of water to cement, sand to cement, cement and mortar to voids, determined (for the required workability) and graphically analyzed as in Fig. 1; the most efficient basic mix selected therefrom; and this basic mix maintained by routine tests of the materials and modifications of the measured proportions (as set forth herein) as the characteristics of the materials vary.

Uniform Quality and Economy—As the exact structure of the mix is maintained, the physical characteristics of the resulting concrete—strength, density, weight, appearance, workability, impermeability, etc., are consistently sustained without a wasteful use of cement. The saving of the margin of excess cement ordinarily allowed in lieu of exact proportioning, covers the cost of this manufacturing control on small, and many times over on large, projects; and, the quality of the concrete is assured before placing. Average savings in cost of materials are as given in Table 3.

The comparative costs of arbitrary-basis and of absolute-basis technical control of concrete production and the comparative outputs per man-hr. of control are as given in Table 4.

Absolute-basis control is based on rates of output per man-hr. of control producing the required quality at the least cost and, while the labor cost is greater than for arbitrary-basis control, resulting economy is as indicated in the following example:

A 10,000-cu. yd. project, requiring 2000-lb. concrete with 1-in. maximum size aggregate, shows the following relative costs:

Per cu. yd. of Concrete	Absolute-Basis Control	Arbitrary-Basis Control	Difference
Cost of materials	$7.09	$7.58	−$0.49
Cost of control	0.24	0.08	+ 0.16
Cost of materials and control	7.33	7.66	− 0.33
Total cost of material and control	$73,300.00	$76,600.00	−$3,300.00
Saving	$3,300.00		

An additional investment of $1,600 for absolute-basis control, returns $3,300 plus $1,600, or a profit of 200 per cent on the cost of absolute-basis control. On larger projects and those requiring higher strength concrete, the economies of absolute-basis control are proportionally greater as indicated by Tables 3 and 4.

TABLE 3—SAVING IN COST OF MATERIALS PER CU. YD. OF CONCRETE

Strength at 28 Days, lb. per sq. in.	Economy of Materials — Maximum Size of Aggregate				
	¾-in.	1-in.	1½-in.	2-in.	3-in.
1500	\$0.44	\$0.41	\$0.38	\$0.37	\$0.35
2000	0.52	0.49	0.46	0.45	0.43
2500	0.66	0.63	0.60	0.59	0.57
3000	0.92	0.89	0.86	0.85	0.83
3500	1.34	1.31	1.28	1.27	1.25
4000	2.06	2.03	2.00	1.99	1.97

(Cement at \$3.00 per barrel and aggregate at \$2.00 per ton at the mixer.)

TABLE 4—RELATIVE COSTS OF ARBITRARY-BASIS AND ABSOLUTE-BASIS CONTROL AND PRODUCTION RATES PER MAN-HR. OF CONTROL

Cu. yd. of Concrete in Project	Absolute-Basis Control		Arbitrary-Basis Control	
	Cost per cu. yd.	Output per Man-hr. of Control, cu. yd.	Output per Man-hr. of Control, cu. yd.	Cost per cu. yd.
1,000	\$0.43	6	9	\$0.21
5,000	0.29	9	18	0.105
10,000	0.24	10	23	0.08
50,000	0.16	16	47	0.04
100,000	0.135	19	62	0.03
500,000	0.09	30	125	0.015
1,000,000	0.075	35	150	0.011

Aggregate Test Requirements—Absolute-basis control depends largely upon the expediency, accuracy and economy of the aggregate tests. Test determinations of the physical characteristics of the aggregates must be systematized, coordinated and simplified to meet the requirements of accuracy, speed, and economy. The following test and calculation methods are designed for these requirements.

TESTS OF AGGREGATES

Determination of coarseness modulus, specific gravity, density, bulking, moisture, absorption, silt and organic matter.

Apparatus:

Container, Scales, and Standard Sieves.

Weight-Volumetric Tests:

(*a*) Weigh water filling container. Empty same.
(*b*) Weigh loose-moist aggregate filling container.
(*c*) Surface dry (*b*) and weigh.
(*d*) Dry (*c*) at 212 deg. F., cool and weigh.
(*e*) Weigh (*d*) inundated, filling container with water.

Supplementary Tests:

(*r*) Make standard sieve analysis finding proportions by apparent volume retained on Nos. 200, 100, 50, 30, 16, 8, 4, ⅜-in., ¾-in., 1½-in., 3-in., etc.
(*s*) Make standard decantation silt test.
(*o*) Make standard organic matter test.

Calculations:

D = Loose-moist density = $(a-e+d)/a$
G = Apparent specific gravity = $d/(a\mathrm{D})$
A = Absorption by apparent volume = $(c-d)/\mathrm{D}a$
M = Moisture by apparent volume = $(b-d)/\mathrm{D}a$
W = Weight of moist aggregate in lb. per cu. ft. by apparent volume = $62.4\,(\mathrm{G}+\mathrm{M})$
C = Coarseness modulus = $1.00 + r_{200} + r_{100} + r_{50} + r_{30} + r_{16} + r_8 + r_4 +$ etc.
B = Bulking of absolute volume by loose measure, moisture and voids = $1/\mathrm{D}$.

WHERE USED AND RESULTS

This method of proportioning has been used since 1924 on projects, aggregating 1,000,000 cu. yd., of every character from dwelling foundations to great engineering structures, including Exchequer Dam and Yosemite Valley Railroad relocation of Merced Irrigation District, Melones Dam of Oakdale and South San Joaquin Irrigation Districts, Pardee Dam of East Bay Municipal Utility District, Oakland-Alameda Estuary Subway ("George A. Posey Tube"), Hunters Point Plant of Great Western Power Company, Sir Francis Drake Hotel, San Francisco, and many lesser projects, all in California.

It has proven thoroughly practical, effected large economies, maintained uniformity and quality of product, established manufacturing methods of control of concrete production, and, is particularly valuable as a basis of exact and comprehensive concrete research.

Concrete for Resisting Sea Water

By Harry E. Squire*

Marine structures are subjected to varying degrees of accelerated weathering. Not only are they immersed in the saline solutions of the sea but in addition they are subjected to severe mechanical abuse which aids the searching sea water attack in uncovering every defect of design and construction. The impinging blows of waves, the repeated wetting and drying of spray, the alternating exposure and covering of surfaces by the tide, the impact of drift impelled by current, the cutting effect of sand, the racking blows of heavy ships—all this mechanical wear and tear acting in conjunction with strong electrolytes of the sea produces a condition where erosion and deterioration is the natural order of things. In fact the destructive effect of the mechanical abuse and the mechanical application of the sea water is so variable that the chemical effect of sea water exposure is largely obscured. The failure to recognize this wide variation in exposure has resulted in confusion in the study of the subject, and to seeming contradictions in the discussion of it.

Irrespective of degree of exposure the deteriorating effect of seawater on concrete structures manifests itself in two general ways:

First, in the retrogression and breaking down of the concrete itself as manifested by softening, cracking and progressive peeling of surfaces. This phenomena is usually attributed to chemical changes in the set cement and to the leaching out of slightly soluble ingredients. Calcium hydrate in the interior mass of the concrete is usually considered the offending constituent on account of its solubility and its tendency to react with sulphate solutions in the sea water.

Second, in the cracking and spalling of the concrete resulting from the corrosion and expansion of imbedded steel reinforcement. In contrast to the behavior of reinforced concrete under atmospheric exposure, this phenomena is attributed to the stronger electrolytes of the sea and to the fact that most of them are unaffected by the concrete, whereas the common electrolyte of the air, carbonic acid, reacts with and is neutralized by the calcium hydrate of the set concrete.

The first type of deterioration is confined to portions of the structure actually immersed, or from the tide line down. As a coincidence the second type or that due to corrosion of the steel affects only that portion above the water or from the mean tide up. Apparently there is insufficient circulation of oxygen in the water to support rapid corrosion.

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The durability of concrete structures in resisting these attacks is dependent quite as much on design as on quality of concrete. Nevertheless the design must be predicated upon the use of a suitable concrete. As the exclusion of water and air from the interior mass is of paramount importance, the most desirable property of sea water exposed concrete is resistance to the absorbtion and circulation of water. Of equal importance is uniformity in the quality of the concrete. Sea air and water are impartial in their attack and any defect or weakness is inexorably disclosed. In this respect a structure in which stability and life depend primarily on resistance to weathering differs from one dependent on resistance to loadings and stresses. A healthy factor of safety may protect the latter against great fluctuations in quality of concrete. But in the former case the action is cumulative; the weaker portions not only yield progressively but they act as foci opening the way for attack on more resistant material. No factor of safety will prevent the sea water from attacking an 800-lb.-per-sq.-in. layer of laitance and sand which might last indefinitely in ordinary structural designs.

Compressive strength is generally accepted as the criterion for measuring quality. But while highly non-absorbent concrete shows a reasonably high strength it does not follow that high strength concretes are inherently the most suitable for seawater work. This is particularly true where strength is achieved by reducing the water content or by increasing the size and quantity of rock to such an extent that the mix becomes harsh or "tricky." Resistance to absorption is greatly affected by the cement content itself and a reasonably high cement content is an essential requirement for sea water resistant concrete whether located below or above the water line. Experience and experiments with briquettes stored in sea water for fifteen years have indicated no retrogression in mortars proportioned one to one and one-half where the sand was given a variable grading up to ten mesh maximum. With well-graded aggregates up to 1½ in. maximum, proportioned after the manner of Fullers curve, a satisfactory concrete can be obtained with a cement content of one to five parts total aggregate. This ratio of cement to total aggregate should be increased when smaller maximum size or other peculiarities of grading require an increased mortar content.

The period of excessively wet mixes is happily passed, and today anyone advocating the use of water is liable to lose caste. Nevertheless sufficient water should be used to produce a mushy consistency still having the necessary stickiness to prevent separation during delivery. On our work a fixed consistency is not specified nor required, but the water is varied slightly to suit the peculiarities of the pouring. Measured by the slump test the consistency ranges around 3 to 5 in., and the concrete gives a 28-day strength of about 2500 lb. The vogue of producing high strengths by employing harsh mixes of low water content in marine structures is almost as dangerous as that which preceded it—of flooding the concrete until it flattened into place without tamping. The use of

mushier mixes with the sacrifice of some strength to uniformity is preferable.

Aggregates for seawater-resistant concrete should obviously be inert to seawater reaction and be composed of non-absorbent material. But in addition they should be selected with a view to producing uniformity when delivered to the forms. The use of aggregates which have a tendency to separate owing to grading, shape or nature of particles should be avoided, and within reasonable limits the several sizes in which the aggregate is delivered and measured should hold their grading without fluctuation. Gravels segregated by washing over screens are generally preferable to crushed rocks which are screened dry and contain varying accumulations of rock dust. The accumulations of fines are particularly annoying as they materially change the cement content of the mortar. On the other hand the practice of adding a small fixed quantity of rock dust, fine sand or diatomaceous earth to produce stickiness and increase uniformity is beneficial if carefully regulated. The use of hydrated lime for this purpose is not desirable in seawater concrete as the calcium hydrate is not inert.

The manipulation of the plastic mass after mixing is as much a factor in the producing of a uniformly resistant concrete as the selection and proportioning of the various constituents. In fact it is possibly the more important factor as the larger part of the technologists' work consists in selecting and proportioning ingredients with a view to facilitating manipulation. At any rate faulty mixes and consistencies may frequently be corrected by laborious care in placing, but no amount of technical skill in selection of materials and proportioning can offset indifferent handling after mixing. The segregation of mortar and aggregate during depositing and the failure properly to compact the plastic concrete accounts for most of the defects observable where the concrete itself has failed in seawater. In order to insure careful placing it is our practice to specify the number of tampers in terms of yards of concrete placed per hour—ordinarily one tamper to two yards per hour. These men are employed solely in agitating and compacting the concrete, and the consistency is adjusted to the tamping specified. The delivery of concrete faster than it can be compacted needs to be guarded against and for this purpose delivery in containers is preferable to delivery in chutes as each dump can be held back until the previous deposit is properly placed.

Construction methods which require speed, driving or other pressure on the men should be avoided. These conditions result from the use of risky cofferdams, working between tides and other methods which are inherently defective. The rapid placing of long vertical depths is objectionable as it is impossible to regulate the accumulation of water or to force out the entrained air and water. But the most objectionable of all construction methods is the placing of concrete under water. When enclosed in a protecting skin of steel or precast concrete such concrete may be relied upon for sealing and for low compressive stresses, but such

concrete should not be relied upon to resist seawater weathering on account of the variability of the product.

In contrast with the objectionable construction expedients cited, the precasting of members is particularly desirable because of the greater facility with which the plastic concrete can be placed and the greater uniformity of the product. The atmospheric exposure of surface is beneficial as the calcium hydrate of the immediate surface is changed to the carbonate which is inert in sea water. In good concrete, however, this skin is exceedingly slight and the great advantage of precast concrete lies in the improved quality and uniformity of the entire mass, rather than in the surface treatment.

Portland cement concrete proportioned as described and placed with the precautions indicated is satisfactory for seawater work. Such concrete will have a total absorption of from 4 to 5 per cent by weight after drying in an oven which resistance appears sufficient to prevent injurious circulation of water after immersion. It has been our custom to increase the fineness of grinding of the cement and to limit the magnesia content to 3 per cent but there is no actual proof that these changes from the standard specifications are essential.

But while concrete of this quality is of itself resistant it cannot be relied upon to protect steel embedded at depths used in ordinary reinforced concrete construction against the corrosive action of seawater. At depths of from 4 to 6 in. a protection equivalent to the durability of the concrete itself may be obtained, but such depths of embedment preclude the use of reinforced concrete as it is ordinarily designed.

On the other hand the damage resulting from this type of deterioration is generally exaggerated. In the first place, corrosion of steel accompanied by swelling and cracking does not occur below the water line so that the damage is limited to the accessible parts of the structure. In the second place, the damage is confined to the outside shell of weatherproofing concrete. In good concrete and under ordinary exposure it makes its appearance as an occasional crack after 8 or 10 years, but does not seriously affect the stability of the structure under 20 to 25 years. Consequently the reinforced concrete structure as ordinarily designed and under average intensity of exposure may be expected to give a service life of 25 years without extensive repairs. Expedients for improving the ordinary design should therefore be measured by this yard stick. In spite of the idealistic appeal of permanence, the earning value of expenditures should control engineering design. With interest figured at 5 per cent per annum, the present worth of a fund for replacing the structure in 25 years is less than 30 per cent of the first cost, and absolute permanence as compared with an assured life of 25 years is worth an additional first cost of about 42 per cent. Since marine structures of reinforced concrete comparable in purpose and capacity are showing differences in cost of several hundred per cent, it is evident that many designers are expending more than is warranted in seeking that will-o'-the-wisp, permanence.

Methods of Prolonging Life of Marine Concrete:

Expedients for prolonging the service life of reinforced structures may be grouped according to method of protecting the steel.

First are the expedients of design intended better to utilize the protective value of the concrete itself. The increased embedment of the steel over that adopted in ordinary weatherproofing is of course the basis of these efforts. The pressure exerted by small diameter bars is less than that of large bars and the spalling can be greatly reduced and possibly prevented by reducing the diameter of the bars used. In our practice the clear covering is made at least 3 diameters of the bar for exposure above the water line. Beams and girders are particularly susceptible to cracking on account of the heavy concentration of bottom steel and the angular shape, whereas slabs and smooth surfaces are more resistant. In addition flat slabs and one-way slabs with wide shallow beams are designed with the heaviest steel concentrations at the upper side of the deck. Increased protection may be accomplished by bending in or splicing the steel where it is not required as in the upper ends of piles above the water line. The cracking of the protective coating under occasional high stresses offers ingress to air and water and can be reduced by carrying impacts of ships axially by means of piles or other members instead of relying on the straining of the structure in bending.

Second are the expedients which seek to seal the concrete against the absorption and circulation of water and air. It appears impossible to accomplish this merely by improving the quality of the concrete. Surface coatings obviously offer a simple method and for this purpose asphaltic compounds having sufficient viscosity to heal over cracks and adequate thickness to exclude air and moisture have proved satisfactory. An uncut asphalt livened with 10 per cent China wood oil and applied hot is used in our practice. A more elaborate process of treating by immersion in asphalt and injection under heat and pressure has also been developed, but it has the disadvantage not only of greatly increased cost but also of non-applicability to the deck structure which is most vulnerable to disintegration from corroding reinforcement.

Third are the expedients intended to protect the steel directly either by sealing and insulating it or by coating it with a more resistant material. Paints and lacquers proposed have for the most part been rejected because of their deleterious effect on bond. The encasement in metallic coatings particularly zinc reduces the bond slightly and does not prevent electrolytic action, but the products of the corrosion of zinc appear to diffuse through the interstices of the mortar without creating the pressure produced by the corrosion of iron. The use of galvanized reinforcement is increasing commercially and so far has proved successful. The recent introduction of cadmium as a coating may also prove successful. Both zinc and cadmium are electro positive to iron and consequently retard corrosion, whereas metallic coatings which are electro negative are undesirable and are known to accelerate corrosion.

Many variations of these preservative expedients are in use and

structures constructed today give promise of greatly outlasting the earlier reinforced structures of twenty years ago. The advisability of resorting to them is largely a problem of the economics of design.

Conclusion—In summarizing the present status of the art as applied to seawater exposed construction, it may be stated with assurance that the methods of ordinary good practice will produce a concrete so durable in seawater that for the engineer's purposes it can be considered permanent. Durability is obtained not through the chemical inertness of the cement itself but through the mechanical resistance of the concrete to the circulation of seawater in the interior mass. It is possible that little more can be effected by the engineering technologist through his proportioning and manipulation of the concrete, but it is probable that much may be accomplished by the chemist in producing a cement less susceptible to chemical change in contact with sulphate solutions, or of a blended cement in which ingredients like ground puzzolan are introduced to react with and render inert ingredients affected by seawater. But in view of the satisfactory results achieved with the present standardized portland cement, such development will be advantageous only in allowing the use of poorer mixes and in offsetting the results of poor workmanship.

The same assurance cannot be expressed in regard to the durability of reinforced concrete. A great deal remains to be effected by the engineer; first, in frankly acknowledging the limitations of concrete in protecting steel exposed to the electrolytes of the sea, and then in educating owners to the necessity of maintaining these structures by periodic painting and occasional repairs. No obloquy is attached to structural steel bridge construction because of maintenance against corrosion by regular inspection and painting. The advisability of similar treatment for reinforced structures is not obvious because engineers and owners have come to regard concrete itself as a reliable protection for steel. Nevertheless, maintenance by painting with impervious coatings offers an inexpensive and reliable means of prolonging indefinitely the life of reinforced structures exposed to seawater. Progress in constructing more resistant reinforced structures must come in large part from the metallurgist in developing non-corrosive coatings for the steel. The galvanizing of reinforcement is being justified with increased use, and the development on a commercial scale of metallic coatings gives more promise of solving this problem than the possibility of securing absolute protection from the concrete itself.

Discussion—Concrete for Resisting Sea Water

P. H. Bates—There is one paragraph in this paper and one heading that I think we should look at the second time. The conclusion to that paragraph is "this will o' the wisp permanence." Many inquiries come to my attention directed toward this matter of permanence, and tracing these back, I find that many are due to a psychological reaction on the part of the user that has resulted from the indiscriminate use of the word "permanence." For instance, he may put down a wooden floor or one of some other material and dig it up. When he put it down, he knew he was going to dig it up, but with a concrete floor it is different. When he finds that he has to repair the concrete floor, it is a tremendous disappointment, because, through this psychology, he has thought that he put down a floor that would be there forever. Mr. Bates.

The author of this paper brought before this Institute for the first time the suggestion that we forget the idea of too much permanence. He has called it a will o' the wisp and he suggests that that point should be emphasized more than it has been.

If it is once realized by prospective users that concrete is *relatively* more permanent than competitive materials, they will begin to consider cost of replacement right in the beginning and will not be so tremendously disappointed when they come face to face with it later.

I think that paragraph should be read over many times, and also the heading on the next page, which, to me, is most significant. He does not say "methods of securing permanence," but simply "methods of prolonging life."

L. W. Walter—I will say a very few words on this subject which Mr. Bates thinks is of considerable interest. A few weeks ago I wrote a specification for damp proofing concrete masonry, with the thought that we might in some cases apply protective coatings. In the specifications, I stated that permanency in concrete is a relative term. Nature will, in time, destroy anything which man can build. To be reasonably permanent, concrete must be made watertight. I think that men in the field possibly have too much confidence in their ability to make concrete of durable quality, and I wrote this feature into the specifications with the thought that it might be an occasional reminder to them that the best they could do in making concrete would not be any too good. Mr. Walter.

Kristen Friis—In my country, Norway, we have a coast that is very long, 1600 miles, and naturally concrete in sea water is a very important question. In investigating the deterioration of concrete Mr. Friis.

in sea water we have secured a series of reports from engineers along the coast. The main result of these reports is to note that all the concrete under water deteriorates very little, but that deterioration takes place in the area between the high water and the low water. Most of our services in sea water are old because the officials of the harbor department for many years have used no concrete in sea water because they did not dare to do it; they have used natural stone.

I think that an investigation may bring forth that the deterioration between high and low water comes from evaporation from the surface, causing hair cracks through which the water will enter to form a sulphate which combines with lime to make gypsum. This causes larger cracks and finally serious deterioration.

In the last ten years, private contractors have built a lot of sea water work in Norway with ordinary portland cement, with excellent results. The most of this work consists of columns under water. Divers prepare the foundations and set up forms which are as watertight as possible. The concrete is always poured through 4 or 6-in. pipes arranged so that the end of the pipe is always below the surface of the concrete. Thus the concrete as it rises from the mouth of the pipe lifts the surface up to the top, so the only part of the concrete that is in contact with the water is this surface. It is not allowed to use any leaner mix than 1:2:2. I know from experience that the consistency of the concrete is very important in curing under sea water.

The best concrete I have ever seen was cured under water in sea water construction. It was made about ten years ago for a coffer dam, thirty feet high, for one of the locks in the harbor of Stockholm. It was absolutely watertight, and we took specimens of that concrete afterwards and polished it and it looked as white as granite. I think that curing under water, if done properly, is the best way of curing concrete.

As to other cements and coating materials, we have tried in Norway in the last year to make a new cement particularly suited for sea water work. It is a Danish invention originating because the Danish people for many years have used with excellent results about 25 per cent of what they call molay, as an admixture to concrete. In this new cement we grind this 25 per cent molay with 75 per cent of ordinary portland cement, and we are hoping that this cement will show better resistance against sea water.

As to coating materials, I think that the most of these coating materials on the market advertised to make the concrete watertight are not very good, because all of them are dissolved in some sort of benzol or other materials, that, when you paint them on, evaporate and leave voids. On the other hand, I think that asphalt, coal tar and other coverings that are boiled on the spot, so that you have no voids in them and no materials to evaporate, perhaps will be good.

I thoroughly agree with Mr. Bates that this slogan of "concrete for permanence" is a relative expression and that we have to protect our concrete in many ways, in better ways than we have used today.

R. J. WIG—There is probably no place on the American coast where there is more high-grade reinforced concrete than in San Francisco harbor. I do not know, however, of any reinforced concrete on the American coast south of San Francisco or south of Norfolk, Va., that is fifteen years old in which they have not had trouble with corrosion of the reinforcing above the high water mark. Therefore, I am sure that the conditions applicable at San Francisco are not applicable to all other locations, and the location must be taken into consideration in attacking this problem of corrosion of the reinforcing steel. Mr. Squire states that it is preferable to add 4 to 6 inches of covering to the steel, and I think he is justified from the experience we have had. But there are several structures, for instance some of the reinforced concrete ships built in 1918 and 1919, that have been in sea water for 10 years, in which the steel is imbedded only ⅝ in. to 1 in. and only recently I have taken pieces out of these ships above the high water line showing no corrosion of the steel. Mr. Wig.

I think there is much that we do not know about this subject, and I think it is possible to get greater durability with our reinforced concrete in sea water than we have been getting. I hope that those who have opportunity of having contact with some of these structures that were built during the war will follow them closely and see if we cannot apply some of the methods used there to some of our structures.

D. V. HAEGERT—On the subject of protective coatings, I want to mention that the Santa Fe Railroad recently has been trying out, in its icing plants along the system, the use of lacquers on concrete, and they seem so far to show very good protective qualities. However, they have not been in service long. Mr. Haegert.

F. MAURO—May I ask what is the action of mollusk life on the surface of a concrete? We know that most of the shell mollusks leave a solution of some salts; eventually they will take some of their food from the surface of the concrete. Mr. Mauro.

R. J. WIG—They have no deleterious action. We have not had an opportunity of observing the portion of the concrete ship below low water, except during the war and immediately after the war. None have been docked in the last eight years, so we do not know their condition under water. However, we do know that on other structures between high and low tide and just below low tide, that there is no deleterious effect; in fact, I would say that it added protection to the concrete. There is one structure on the southern Pacific coast at Huntington Beach, which I believe is the oldest concrete structure south of San Francisco; all the others have failed. This one was built in 1911 and is still in fair condition. Mr. Wig.

KRISTEN FRIIS—In regard to concrete ships or barges I have had an opportunity to inspect five of them in the last year, and I must admit that they are in absolutely excellent condition. They were built in the years 1915 and 1916, so they have been in service now for thirteen years. These barges were built with the keel upside down, used very heavy reinforcement and a 1:2:2 mix. These barges are so absolutely watertight Mr. Friis.

that one of the cement manufacturers in South Norway has used them for many years as storage bins. The pressure on their bottoms is about fifteen feet, which indicates that portland cement concrete in sea water will stand up.

Mr. Price. Phillip Price—I think it has been brought out conclusively by the paper and the discussion that the critical point of concrete structures in sea water lies between the low water level and the high water level. I want to point out that that also applies to fresh water structures, such as bridge piers in certain localities. In Pittsburgh when we began to build concrete bridges it was noticed that deterioration did take place at or near the water line, and to overcome that in recent years, we have adopted the plan of facing our piers with either sandstone or granite for a distance of 2 ft. above and 3 ft. below the low water level. We find that that is a very satisfactory solution, both for freezing and thawing and that it also protects the concrete against the acid condition that exists in our rivers, which in some cases is worse even than sea water.

Mr. Wasson. J. H. Wasson—In line with what has just been said, I would like to point out that the "water line" is the danger line regardless of what material is used. Steel, wood, limestone, and granite deteriorate first at the water line. Since we are forewarned by observation and experience that the water line is the danger line, we can safeguard the future life of the concrete structure by making the concrete at that point of the very highest quality it is possible to produce. This is not possible with the other materials mentioned. We have the opportunity of saying that the concrete at that particular point shall be of a very definite mixture, and made with the very best materials, placed without segregation, and that there shall be no end of days run at this point. The concrete at this point must be as watertight and impermeable as we can make it. With a more wide realization that the water line is the danger line with concrete structures, we believe that engineers are now beginning to take the above simple precautions which will prevent in the future much of the trouble with concrete at the water line.

Mr. Wig. R. J. Wig—I wish to point out that the water line is not the danger line for reinforced concrete in sea water. The danger line is just above the high water line, and the corrosion of the reinforcing steel in sea water is always started above high water and not between the low and high water mark.

Mr. Wasson. J. H. Wasson—When I said the water line, I meant to include, for concrete, the belt from 2 ft. below to 2 ft. above the water line; certainly the point where the water is absorbed in the concrete and comes up and evaporates and produces the result referred to.

Mr. Squire. H. E. Squire (*By Letter*)—The writer is grateful to Mr. Bates and Mr. Walter for emphasizing the need of a modified viewpoint in regard to the current acceptance of "permanence" as an inherent characteristic of concrete. This phase of the discussion may appear irrelevant in a paper dealing with the properties of concrete for sea water exposure. Nevertheless if permanence is to be prescribed as a necessary requisite for all con-

crete, constructors may as well give up its use in sea water construction. And this goes for other types of construction such as roads and pavements where the material is subjected to wear and tear and progressive shattering. The damage resulting from this unfortunate viewpoint is not confined to the embarrassment of engineers and constructors; it has a more important aspect in misleading the business community and the public. Inadequate allowances for depreciation because of structures "built to last forever" and 75 year bond issues for structures having a reasonable life expectation of half that period are tangible evils.

Mr. Friis' discussion of the status of sea water work in Norway is instructive because it presents an independent practice remote from the influences which tend to standardize methods in this country. Three interesting features brought out by him are the marked disintegration of concrete between the tide lines along the coast of Norway, the apparently successful use of tremie concrete, and the introduction of admixtures for increasing the resistance of the concrete. The writer has touched on the very great variation in intensity of exposure and its effect in confusing this subject. Undoubtedly disintegration in the more northern latitudes is influenced and accelerated by subjecting the wet surfaces to freezing temperatures. Mr. Wig likewise touched on the same theme in noting the effect of accelerated rusting in southern latitudes. Possibly San Francisco is blessed with the ideal mean between these two disintegrating influences. The success of tremie concrete operations would seem to belie the writer's castigation of this expedient. Resort to this method of placing concrete reminds the writer of the old time mountebank who advertised his show by driving full speed around Main Street blindfolded. One had to admit that he actually did it; but that didn't make it safe for the ordinary citizen to try and besides it could be done much better by anybody not blindfolded. In regard to admixtures, the writer thinks they should be used with extreme caution; and, as stated, that a pozzolanic admixture for absorbing the free lime of the concrete appears to have merit. But in view of the excellent results obtained from standard portland cement, we would hesitate to introduce the same in important work until proved for ten years or more in minor work.

The most illuminating remark of Mr. Friis, however, is the following: "Most of our services in sea water are old because the officials of the harbor department for many years have used no concrete in sea water because they did not dare to do it." In other words, having once been "stung" they discontinued the use of this material. In this connection, it is interesting to note that during the past 39 years since concrete was introduced in San Francisco Harbor construction, there have been no less than three reactions against the use of concrete, resulting from the disillusionment of constructors and officials when structures intended "to last forever" began to show evidences of decay and the need of repairs. And in our case the Commission did not turn toward a more nearly permanent material such as stone masonry but to a less resistant material, namely treated timber, with the definite idea of anticipating and providing for replacement.

Mr. Wig's very complimentary references to work at San Francisco might incline some to think we are producing a sort of super-concrete. On the contrary, our efforts are concentrated not on perfecting our concrete technique but rather in obtaining maximum durability for the money expended. Better and also more expensive concrete is installed in many places, and it is a mooted question with us as to whether the increased expense of additional cement, elaborate finishing and special processes is warranted by the increased durability obtained. This is illustrated somewhat in the references to concrete ships, the condition of which in the writer's judgment does not fully substantiate Mr. Wig's optimism. An inspection of two vessels moored in this bay for seven years indicated that practically the entire ship except in the vicinity of the water line was in perfect condition, but close observation of the belt immediately above the water line showed numerous line cracks, evidence that the typical corrosion had started at this vulnerable location. This concrete was proportioned one to three or better of specially ground cement, the surfaces had been carefully smoothed and treated with spar varnish and asphalt. Yet neglect in maintaining a suitable coating had apparently negatived the increased expenditure and reduced the resistance to about that of a good standard concrete under the same exposure.

In regard to coatings, the application of hot asphalt has been adopted because it is the cheapest coating obtainable which will seal the surface, but to accomplish this it should be at least one sixteenth of an inch thick. Asphaltic coating is not practicable below the water line, because the mollusks (inquired after by Mr. Mauro) and other marine growth break it off and detach it from the concrete. It is also subject to rapid deterioration when exposed to sunlight—even to the reflected sunlight at the edges of the structure. But underneath the decks of open piers and wharves where sunlight does not reach, the coating appears to be good for at least 5 years. Asphaltic coatings of course are not applicable to locations where appearance is a considerable factor; but as we have comparatively little of this exposed to sea water we have never developed a light colored coating, leaving the same to a field of industry which is even more complex than that of producing desired results in concrete.

In concluding may I add that the one thing that justifies the use of concrete in any particular sea water structure is that in the long run it happens to be the cheapest material available. The principal competitor of concrete for this type of service is not the more nearly permanent stone masonry construction with its prohibitive cost but the less expensive and shorter lived treated timber construction. The writer believes that progress in adapting concrete to marine structures should be pointed toward effective utilization of the material with a view to reducing cost rather than toward the perfecting of the material with a view to producing permanence.

Concrete Exposed to Alkali Ground Waters

By C. J. Mackenzie, M.E.I.C.*

In discussing the general subject of concrete exposed to alkali ground waters it must be admitted at the outset that at the present time no simple answer can be made to the query: "How can concrete be made immune to alkali attack?" And while it will be generally admitted that under alkali conditions of the most extreme nature it is not safe to have concrete even of first class quality freely exposed, it should also be realized that the extensive investigational work that has been carried out on this continent during the past 20 years has not been sterile, but that advances of real value have been made, which serve as a guide to those who find it necessary to build structures exposed to alkali waters; and that, furthermore, the advances already made give promise of bearing the germ of a complete solution of the problem.

It is proposed in this paper to note briefly the progress made in this regard, more especially within the past few years.

The fact that sulphate waters, such as are known quite generally throughout the semi-arid areas in Canada and the United States as alkali waters, will attack portland cement concrete has been known for many years, and over 100 years ago L. J. Vicat of France propounded a theory to explain the disintegration of hydraulic cement by sea water.

On this continent, however, it was not until about 25 years ago that such disintegration came generally to the attention of engineers, and the question was then raised as to whether or not well-made concrete would resist alkali action, and while the early laboratory investigations at Montana State Agricultural College,[1] the University of Wyoming[2] and at other places indicated that alkali ground waters will attack portland cement mortar, for a long time many engineers of standing maintained that a well-made and dense concrete would not be so attacked, and that such failures as were noted were the results of weathering of inferior products and the weakening by crystallization of salts and ice in porous concrete. By 1921 there were available the results of reliable and extensive field tests[3], [4], notably those started by the Bureau of Standards in 1913 and similar tests in Canada made under the auspices of local branches of the Engineering Institute of Canada, and these results crystallized well informed opinions somewhat as follows:

(1) The highest quality of portland cement concrete is not immune to alkali ground waters of high concentrations.

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(2) The active agents in alkali waters are the sulphates of sodium and magnesium.

(3) The stronger the concentration of sulphate salts in the water the more active the disintegration.

(4) The concentration of alkali ground waters varies widely between points only a few feet apart, and, therefore, a structure has not the same exposure conditions at all points. Also, the concentrations vary widely throughout the year.

(5) A well made, strong and impermeable concrete will resist alkali action much better than a weak permeable one.

(6) Whenever the sulphate concentration in an alkali water is higher than 0.1 per cent even a well-made concrete is in danger if exposed to such water.

(7) Ordinary bituminous coatings delay the action, but the protection is not permanent.

(8) No integral compounds tested gave protection.

(9) Where concrete structures must be built in alkali soils, a high-grade concrete should be specified, and the water should be kept from coming into contact with the concrete either by the use of membrane waterproofing or by draining the site of the work.

(10) The action between the sulphate waters and the mortar is a chemical one essentially, the disrupting forces of crystallization are secondary and the exact chemical reactions are unknown.

Since that time much research work has been done and the two most widely known investigations are those of T. Thorvaldson and D. G. Miller. Thorvaldson's work was undertaken under the auspices of a committee of the Engineering Institute of Canada which felt that a careful and fundamental chemical research into the nature of the reactions of disintegration would ultimately prove most profitable. Miller's work has been done as part of a co-operative investigation conducted by the U. S. Department of Agriculture, the Department of Drainage and Waters of the State of Minnesota and the Department of Agriculture of the University of Minnesota, in connection with the use of drain tiles in alkali soils. These two investigators, one a chemist using the technique of the physical chemist, the other an engineer using the methods of a research engineer, have supplemented and verified very important findings of each other.

Today we know a great deal more about the problem than we did in 1921 and although for mass concrete work it is not as yet possible to give specific advice that will guarantee immunity from alkali attack, much better general advice can now be given than was possible then.

The significant findings of the past few years are as follows:

(1) It has been found that the actions of sodium sulphate and magnesium sulphate, which formerly were supposed to be similar differ materially[5] and affect different cements in different ways.

(2) It has been found that concretes and mortars made from portland cements from different mills and from different raw materials may differ

widely in their stability in alkali waters, and that under the same conditions of exposure it is possible to select two cements, the concrete made from one of which may be completely disintegrated in, say, two years' exposure, while concrete of comparable mix made from the other may be very little affected in that period of time[6, 7].

(3) It has been found that strength tests are not indicative of the relative sulphate resistances of different cements. But it should be noted that, speaking generally, when using any given cement the concretes giving the greatest strengths are the most resistant for mixes up to say 1:2. But it was found that neat cement blocks on the other hand disintegrate in a very unusual and violent manner[7, 8].

(4) It has been found that some high alumina cements are very resistant to attack by magnesium sulphate and are more resistant to sodium sulphate than most portlands but, on the other hand, they are quite easily attacked by sodium carbonate, which is present in many localities in the West in the form of so-called "black-alkali."[7]

(5) It has been found that, if even a lean concrete made from a cement of low natural resistance to alkali be steam-treated at a temperature not lower than 212 deg. F. for a sufficient length of time, it can be made absolutely immune to attack by sodium sulphate and very resistant to attack by magnesium sulphate, and it is concluded that if a rich, strong, impermeable concrete made from a cement of high resistance to alkali be so steam-treated, it will be not only immune to attack from sodium sulphate but will be also practically immune in magnesium sulphate waters. It was also found that if the temperature be less than 212 deg. such resistance is not developed and under certain conditions the resulting product may be made even less resistant[6, 9].

It is felt that the above findings, may have an important affect on future engineering practice, providing the chemists are not able to produce a modified portland cement capable of resisting all alkali action, a product which we still hope may materialize.

It is easily seen that for concrete pipes, such as sewers, drain tile and any pre-cast work it is now possible by proper steam treatment to produce a product that will probably be satisfactory for all normal sulphate exposures. It remains now only to work out the economic balance with respect to steam pressures and temperatures, time of treatment and other factors.

There still remains the problem of mass concrete work exposed to alkali waters and unless steam-treated pre-cast slabs are used for sides and bottom, steam treatment seems to have nothing at present to offer. Certain effective precautions, however, may be taken and these will be outlined as follows: First, the ground water should be carefully analyzed to determine just what salts are likely to be present and in what proportion and concentration. Second, a cement should be selected that shows a high resistance to that type of water, and while at present no simple, easily made test is available for such determination, it is quite possible to do this by the methods of either Thorvaldson[7] or Miller[8] and it is

hoped that before long some approximate method, suitable for the ordinary testing laboratory, may be developed. Third, all the well-known steps should be taken to obtain a rich, strong impermeable concrete.

If the above precautions are taken, under mild alkali conditions a long life for the structure may be had, and under any condition the life will be greatly increased. If, however, the sulphate concentration of the ground water is high, say, greater than 0.5 per cent such precautions will probably be found insufficient, and the concrete must be protected by membrane waterproofing or else the site must be drained so that all water will be prevented from coming into contact with the concrete.

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[1] Bulletin No. 81, Montana State Agricultural College.

[2] Bulletin No. 122, University of Wyoming.

[3] Technologic Paper No. 214, Bureau of Standards, Washington.

[4] "The Disintegration of Concrete."—Williams. *Engineering Journal*, August, 1921.

[5] "Concrete Deterioration in Alkali Soils."—Mackenzie. *Engineering Journal*, November, 1925.

[6] "Action of Sulphate Waters on Concrete."—Miller, D. G. *Public Roads*, November, 1927.

[7] "Notes on Relative Resistance of Various Cements to the Action of Sulphate Waters."—Thorvaldson. *Engineering Journal*, March, 1928.

[8] Journal Series Paper No. 625.—Miller. University of Minnesota.

[9] "The Effect of Steam Treatment of Portland Cement Mortars on Their Resistance to Sulphate Action."—Thorvaldson. *Engineering Journal*, March, 1928.

Discussion—Concrete Exposed to Alkali Ground Waters

R. J. Wig—I would like to ask Mr. Bates whether the steam curing of the concrete makes it immune? Mr. Wig.

P. H. Bates—I can conceive such to be the case under certain conditions. We hear of this material referred to as jell so much in connection with the discussion of cement, and we know that certain jells under certain conditions, with certain treatments, can be rendered more or less immune to the further action of water. We do not have this definite information in connection with this hypothetical jell that exists in portland cement. But we do know that in other cases, other types of this same noncrystalline variety of matter when subjected to certain temperatures and atmospheres (with regard to humidity) do have a change produced in their character so that they will not later absorb the same amount of water as they would before. In the case of these steam treated cement products, you might say, that the treatment has resulted in this colloid being rendered irreversible. If it had not been so treated, we would have the condition existing that, on placing the concrete in water, it would take up a certain amount of water. However, on submitting this same product to a dry atmosphere, it would give up a large part of that water and that cycle could be repeated back and forth indefinitely. We say that there are certain types of these colloids or jells which, under certain treatment, are rendered irreversible—a typical example of which is the white of an egg; you can readily mix it before you have heated it, with a large quantity of water, but after heating it, it is irreversible and is no longer affected by water. We can conceive this same condition as existing in portland cement products. Mr. Bates.

Special Characteristics of Concrete for Pavements

By F. H. Jackson *

Taking everything into consideration, pavements are called upon to resist probably not only as severe but also as great a variety of destructive forces as any type of structure in which this versatile engineering material—concrete—is used. It is the purpose of this paper to discuss the special characteristics which pavement concrete must possess in order to offer the maximum resistance to these destructive agencies as well as to indicate how these special characteristics are affected by the materials and methods of fabrication employed. The paper will deal entirely with the quality of the concrete itself and will not touch upon the structural design of the pavement slab which is a separate though related problem.

In discussing the forces which operate to destroy our concrete pavements, it will be interesting to consider first the kind of structure with which we are dealing. How does it differ from other types of concrete structures? In what respects is it unique? A concrete pavement, from a structural standpoint, consists of a series of flat slabs several feet in width, only a few inches in thickness, and of indefinite length, resting upon a support of more or less uncertain character and subjected not only to traffic loads of greatly varying intensity but also to the stresses and weathering effects produced by wide ranges in temperature and moisture conditions. Few concrete structures are subjected to such a variety of destructive forces and in order to afford the greatest possible resistance to all of them the utmost efforts towards securing the best possible product are justified. The fact that so many of the concrete pavements built years ago are still carrying traffic at a reasonable cost of maintenance is a tribute to the inherent worth of concrete as a paving material. The fact that so many of the concrete pavements built within recent years have failed to measure up to the high standard of service required of them should serve as a warning that in so far at least as pavements are concerned the very best engineering control both as regards design and construction is absolutely necessary if satisfactory results are to be obtained.

Destructive Agencies

Destructive forces continually at work upon concrete pavements and to resist which the material must be designed, may be divided into two general classes—first, the forces due to natural causes, such as variation in temperature, moisture, etc., and second, forces due to traffic loads.

* Senior Engineer of Tests, U. S. Bureau of Public Roads.

Natural forces produce not only direct tensile and compressive stresses in the concrete but also the complex stresses or "weathering effects," which sometimes result in partial or complete disintegration and to resist which we strive to produce what we term "durable" concrete. Direct stresses due to natural causes are in general produced in the concrete because of the resistance offered by the subgrade to the free expansion or contraction of the slab and may be induced either by changes in temperature or moisture content or both. Such effects manifest themselves from the moment the concrete pavement is laid and continue as long as it is in existence. They become evident shortly after the pavement is constructed through the formation of transverse contraction cracks due to the drying out of the slab—unless the pavement is provided with construction joints spaced sufficiently close so that the maximum tensile stress due to drying out is less than the tensile strength of the concrete at the time contraction sets in. Much could be written regarding the necessity for continued and adequate curing of concrete in structures of this type in order that its tensile strength may be built up to a maximum before the pavement is allowed to dry out and tensile stresses thereby induced which tend to crack it. It should suffice here, however, to indicate that thorough curing, especially during the period immediately following the casting of the slab, should tend to reduce the number of cracks due to this cause, especially those surface cracks or checks produced by the drying out of the surface of the concrete at a greater rate than the mass. Temperature and moisture effects in general manifest themselves in the building up of compression or tensile stresses within the concrete due to the inability of the slab to contract or expand freely. Forces of this character are operating continually during the life of the pavement and the concrete may be said to be continually under stress due to them. Fatigue effects as well as the possibilities of the flow of concrete under sustained load as a factor in relieving stress, are but little understood and should be investigated.

In order to combat these natural forces, the concrete should possess high resistance to both compression and tension. Resistance to compression has at all times been considered an important property of concrete and for many types of structure it has been deemed the most if not the only essential property. However, in the case of pavements, tensile strength, as has been shown, is fully as important as compressive strength. In spite of many opinions to the contrary a crack in a concrete pavement must be considered as a structural defect. This is particularly true in the case of unreinforced pavements because an open crack is in effect an unsupported edge and as such, for a given thickness of concrete, becomes the weakest part of the structure.

It has been shown[1] that a direct relation exists between the tensile strength of concrete and the amount of transverse cracking which will take place in a pavement slab. Other things being equal, the spacing of

[1] A. T. Goldbeck, *Public Roads*, August, 1925, "The Interrelation of Longitudinal Steel and Transverse Cracks in Concrete Roads."

transverse cracks due to contraction will be directly proportional to the tensile strength of the concrete at the time contraction begins.

As the result of much research work by the Portland Cement Association as well as other organizations, the water-cement ratio law governing the strength of concrete has been established. The writer believes that this principle, when correctly applied, affords the simplest and most practical method of designing concrete mixtures for a given strength yet devised. The assumption, however, that simply because the average of many thousands of tests show for instance that to obtain a crushing strength of 3,000 lb. per sq. in. at 28 days all that is necessary is to maintain a water-cement ratio of 0.8 will not serve as an adequate basis of design for paving concrete. This is due to the fact that although crushing strength in paving concrete is important tensile strength and flexural strength are more important and we are not at all sure that there are not other factors fully as important as the water-cement ratio in controlling these properties.

Experiments conducted by the Bureau of Public Roads[1] and elsewhere have shown beyond question that the resistance of concrete to tension is affected by the character of the aggregates employed to a much greater degree than is the compressive strength. However, these same experiments indicate that for *a given combination* of aggregates and cement, the water-cement ratio governs the tensile and flexural strength fully as closely as the compressive strength. The answer to this, in the writer's opinion, is to use the so-called "trial" method of proportioning the application of which to the design of concrete paving mixtures was described in a recent issue of *Public Roads*.[2]

WEATHERING

The slow acting but long continued destructive effects of weathering, frost action, etc., on the concrete pavement, though admittedly of great importance, are still but slightly understood. We speak somewhat vaguely of "durability" as essential in all classes of concrete exposed to the weather or to corrosive action of any sort, but we are not nearly so certain as yet regarding the factors which affect durability as we are regarding those which affect strength. Here again the question arises: Is strength a measure of durability and if not what factors affect durability which do not affect strength? In an effort to throw light on this subject, alternate freezing and thawing tests of concrete are being conducted in a number of laboratories. These tests indicate that although there are certain types of unsound aggregates which will produce unsound concrete even though it may be satisfactory from the standpoint of strength the same factors in general affect durability as affect strength. This applies particularly to the amount of mixing water used, so that given sound

[1] "Comparative Tests of Crushed-Stone and Gravel Concrete in New Jersey," F. H. Jackson, U. S. Bureau of Public Roads, *Public Roads*, Feb., 1928.

[2] "The Design of Pavement Concrete by the Water-Cement Ratio Method," reported by F. H. Jackson, Bureau of Public Roads, *Public Roads*, Aug., 1928.

aggregates the best insurance against frost action and weathering effects in general still appears to be to employ as low a water-cement ratio as possible consistent with the use of a plastic workable mix.

Unsound concrete will of course be produced by using unsound aggregates. Such materials as shale, certain varieties of flint, etc., occurring either in the fine or coarse aggregate must be avoided if sound concrete is to be obtained. It is surprising, however, what a wide variety of aggregate types may be employed and durable sound concrete produced provided it has been properly designed and fabricated. It seems safe to say that any material which will pass the sodium sulphate soundness test[1] should prove satisfactory in concrete in so far as durability is concerned. Conversely, failure in this test should subject the aggregate to suspicion until a thorough field investigation at the source has convinced the engineer that it is safe to use. Control of the aggregates themselves, however, will go for naught unless the proper care is exercised in fabrication, and it is in the failure on the part of the field man to recognize one or more of the fundamental principles of good concrete construction that accounts, in the author's opinion, for most of the failures which we are apt to attribute to lack of durability.

SCALING

The causes of surface scaling of concrete pavements are little understood, although there is considerable evidence to indicate that it is the result of frost action almost entirely. The fact that scaling is not observed in the South, but is confined entirely to the Northern States substantiates this view. It has been possible in the laboratory to simulate the surface scaling of concrete as observed in service by subjecting it to alternate freezing and thawing. Surface disintegration almost always begins on the top of the concrete specimen, and is probably due to the inability of the relatively weak, porous mortar surface to resist disintegration to the same extent as the mass of the concrete. If this view of the case is correct, it would indicate that in order to minimize the danger from scaling the greatest efforts should be made to prevent the formation of such a porous mortar top on the surface of the pavement. The use of a stiff concrete just sufficiently plastic to settle into place without tamping, the use of a fine aggregate containing as little silt or other fine material as possible, and the removal of the thin surface by means of lutes just prior to final finishing should go far towards preventing this type of destructive action. Surface scaling followed by disintegration may sometimes be due to the use of unsound aggregates but it would appear that failures from this cause are more or less local and do not compare in extent to the scaling which may be attributed to one or more of the causes given above.

TRAFFIC LOADS

Undoubtedly the major stresses in concrete pavements are the bending stresses produced by traffic loads applied either statically or by impact.

[1] U. S. Department of Agriculture Bulletin, 1216 Rev., page 20.

These stresses are a maximum, in general, along the edges and at the corners of the slabs and much work has been done in establishing the most economic pavement cross section for uniform slab strength. Although it is not the purpose of this paper to discuss slab design it may be well to indicate that here as in all types of structures the adequacy of the design depends entirely upon how closely the strength of the concrete as actually placed conforms to the strength assumed in the design. In all formulas for pavement slab design, a safe unit flexural strength of about 300 lb. per sq. in. is ordinarily assumed, and the thickness of the slab necessary to carry a given maximum wheel load is calculated on this basis. For a factor of safety of two, this requires the design of a concrete mixture having a modulus of rupture at 28 days of 600 lb. per sq. in., which is equivalent to a crushing strength of about 3,000 lb. per sq. in. As has been noted, the water-cement ratio principle may be utilized in the design of concrete for a given flexural strength provided it is recognized at the outset that such factors as angularity and surface texture of aggregates affect the results to a greater extent than in compression. For instance, tests now being made by the Bureau of Public Roads in which 17 typical varieties of coarse aggregates including trap rock, limestone, granites, calcareous and siliceous gravels and blast furnace slags are being investigated, show that for a given mix, grading and water-cement ratio, the maximum variation in flexural strength of concrete made with these aggregates amounts to as much as 25 per cent of the average strength. An illustration from one of the four mixes employed in the investigation may be cited. The flexural strength (modulus of rupture) of a field volumetric 1:2:3 mix, using a fixed consistency and with the water-cement ratio falling within a total range of .05, varied from 530 lb. per sq. in. to 650 lb. per sq. in. at 28 days, due entirely to the kind of coarse aggregate employed. This difference in strength is equivalent to that which would result from a change in the water-cement ratio of 0.15—a variation which would certainly not be ignored by those who believe in the water-cement ratio law. Furthermore, this change in flexural strength was not accompanied by any significant difference in crushing strength, indicating that even quite wide variation in character of coarse aggregate had no appreciable effect upon compression for a given water-cement ratio. This point is emphasized here simply to show that in designing pavement concrete mixtures other factors besides water content, important though it is, must be considered.

SURFACE WEAR

To just what extent is resistance to surface wear important in pavement concrete, and what factors affect it? Many engineers believe that high resistance to wear is no longer of importance and point to pavements under heavy traffic which were constructed with relatively soft aggregates but upon which the original finishing marks still show. On the other hand, the use of steel skid chains on automobiles in winter must surely produce wear. This is, however, about the only destructive agency

of this type left, since the steel wagon tire has practically disappeared from our highways except in certain restricted regions. On the whole, the writer is inclined to believe that surface wear is not now a critical factor and that in general a concrete mixture which has been properly designed as to strength, durability, etc., will also be satisfactory from the standpoint of surface wear.

QUALITY OF PAVEMENT CONCRETE AS AFFECTED BY CONSTRUCTION METHODS

The writer has had occasion once or twice in this paper to call attention to the fact that quality concrete for pavements depends fully as much upon the care used in construction as upon the materials employed or the proportions established in the design. This point can not be overemphasized. Of what use is the utmost care in selection of the materials and in the designing of the mixture if these are not followed up by intelligent and painstaking control throughout the construction period. The writer has no patience with the type of engineering control which very carefully specifies the quality and grading of aggregates to be employed, sets a mix which under laboratory conditions will give the designed strength, and then employs a farmer's boy as inspector on the job. Such inspection is unfair to the contractor, as well as to the public, because it often leads to arbitrary and unreasonable interpretation of specifications resulting in unnecessary delays and increased cost.

The writer believes that there is also a tendency to overemphasize speed in construction in certain quarters to the possible sacrifice of quality. Maximum efficiency is of course much to be desired in all contruction operations. It must be remembered, however, that mere speed should not be permitted to be the controlling factor beyond the point where it is possible to maintain the highest standards of workmanship. Furthermore, no established practice as regards construction should be modified for the purpose of increasing production without first determining very definitely that the proposed modification does not adversely affect the quality of the finished product.

On the other hand, we should also be alive to the possibilities of improving the quality of our pavement concrete, especially if we can do so without increasing cost of production. With this thought in mind, the Bureau of Public Roads is now actively promoting in connection with the administration of Federal Aid to the States certain principles in connection with the production of concrete for pavements which will, we believe, result not only in more uniform quality but will also tend to reduce the ultimate cost to the public. These principles may be stated as follows:

(1) The abandonment of volumetric proportioning of aggregates and the adoption of proportioning by weight. Inundation will be recognized as a permissible alternate method for fine aggregate but weighing will be preferred.

(2) Maintenance of the lowest water-cement ratio which, with the type, grading and proportions of aggregate and methods of finishing employed, will produce a workable, dense and uniform concrete.

(3) The scientific grading of coarse aggregate by combination of separated sizes in each batch in the proportions which will produce the maximum practical density.

(4) The abandonment of hand finishing methods.

Where adequate engineering control is assured, and on request of the States, the use of a proportion of coarse aggregate greater than the proportion in the approved standard mix will be permitted if by combination of separated sizes in each batch a well-graded aggregate is produced and the resulting concrete is dense and uniform, and workable by the process of finishing employed, and of a quality at least equal to that produced by the approved standard mix.

The advantages of weight proportioning have been clearly set forth by Crum[1] and do not need to be recapitulated here. Suffice it to say that weighing has been demonstrated to be not only a more accurate but also just as efficient and economical a method of measuring aggregates as volumetric proportioning.

By measuring coarse aggregates in two or more separate sizes, the Bureau feels that improvement in uniformity as well as increased economy will be accomplished for the following reasons:

It will insure the maintenance of a uniformly low void content in the coarse aggregate from batch to batch, thus making it possible to reduce the amount of mortar below that necessary under the present practice where quite wide variations in voids occur frequently from batch to batch due to inefficient mixing of sizes at the producing plant or to stock pile segregation. That it has been possible to do this has been demonstrated in actual construction in North Carolina where a proportion containing considerably more coarse aggregate than we have been in the habit of permitting has been found practical through the use of 3 separate sizes of coarse aggregate. Another outstanding advantage of handling and proportioning coarse aggregate in this manner is that it makes possible much closer water control at the mixer through the elimination of a variable which under present practice causes more trouble than is commonly supposed. This is the fluctuation in the water requirements of individual batches due to changes in grading. Batch to batch variations in the quantity of the finer sizes in the coarse aggregate, that is the material ranging from about ¾-inch down, are quite common under the present practice and cause marked variation in the workability of the concrete. This in turn leads to a tendency on the part of the mixer operator to control the workability by changing the amount of water. It will be admitted that uniform water content is essential to uniform concrete. Measurements of coarse aggregate in separate sizes will contribute much to this end.

[1] *Public Roads*, March, 1927, "Proportioning Concrete Aggregates by Weight," by R. W. Crum.

As regards the abandonment of hand finishing methods, the Bureau feels that in general smoother riding surfaces can be produced with mechanical equipment, and also that it is possible economically and efficiently to handle concrete containing a higher percentage of coarse aggregate and also drier concrete than when hand finishing methods are employed. The manner in which a finishing machine will handle a concrete which by all laboratory standards would be labeled absolutely unworkable has considerably altered our conception of what we term (for want of a better name) "workability" in concrete, at least in so far as pavement construction is concerned.

The Bureau believes that the application of the above-mentioned principles to concrete pavement construction will result not merely in increased strength but in greater uniformity, and for this reason are important from an economic as well as a physical standpoint.

The Bureau believes that the loose methods of control which have been the rule in the past have often led to the use of proportions capable of producing a concrete of considerably higher strength than called for in the design in order that we might be certain of obtaining the design strength in the field. In other words, we have been employing a factor of safety in the shape of richer mixtures to take care of inadequate control methods. The greater certainty of the methods proposed should enable us to design and produce concretes conforming more closely to whatever design requirements may be imposed with resulting benefits both physical and economic.

Mr. Mauro. F. Mauro—The speaker has, in a very interesting way, presented to us the desirability of a study of concrete which is used for roads, from the point of view of flexural resistance. That is very good, but why is it necessary? Certainly it is not because of the load we are carrying upon the road. These loads would not be serious if we had a subsoil properly compacted. The U. S. Bureau of Public Roads would do a great service if it were to bring to the mind of the contractor and of the public administrator the fact that it is very important that we do not put a very good surface on a very poor base. A poor base is one of the greatest reasons for road failure. Where we have a proper base we have nothing else but the crushing strength of the concrete to be taken care of.

Now as to the question whether it is better to use weight proportioning or road work: Is it really any different from the volumetric method of measurement? I do not see that it is, because the weight of each aggregate which enters into the concrete is based on its specific gravity. Therefore, when we speak of weight we have indirectly spoken of volume. We may use weight to measure the aggregates, but the volume is the base.

Mr. Burns. A. S. Burns—I would like to ask Mr. Johnson if the information collected during the past year on flexural strengths has been tabulated and is available yet to the public?

Mr. Jackson. F. H. Jackson—That information is expected to be available this summer.

Mr. Allyn. E. H. Allyn—I would like to ask a question in regard to how the weight is determined. For instance, in material yards, where the material will be received in car lots and unloaded in piles in different parts of the yard, there will be rock from Wisconsin in one pile and rock from a variety of other places in other piles. All of these rocks may have a different weight per cubic foot. How would you proceed to determine an average unit weight?

Mr. Jackson. F. H. Jackson—Of course, it is necessary always to investigate the specific gravity of the aggregates and set it accordingly. The heavier aggregates will weigh more per given volume than the lighter ones, but in concrete pavement construction as a rule we do not have to worry about variations in specific gravity after we have finally decided on the material we are to use. In other words, having fixed the weight for the particular material we are to use, there is not very much change in the specific gravity during the course of construction unless the source of supply is exhausted. For instance, if you have a 1:2:3½ volumetric mix specified, you can determine from the weight of materials as delivered

on the job what the weight of say 21 cu. ft. of that material would be. Of course, that is based on certain average specific gravities. The gentleman's point does not apply, because, once having decided upon the aggregate to use, there is very little deviation in the aggregate throughout the construction of a given job.

F. H. MENEFEE—I am interested in the statement that a 25 per cent variation in flexural strengths had been found. I would like to ask if the corresponding variations in crushing strengths are available? Mr. Jackson intimated that the crushing strengths did not vary that much, and that they, therefore, did not give an accurate indication of the flexural value of the pavement. **Mr. Menefee.**

F. H. JACKSON—This paper does not give that information, but it will be released in detail this summer. This paper will contain the crushing strengths, flexural strengths and tensile strengths of concretes using all of the various materials noted. The crushing strength does not vary for the various types nearly to the same extent as do the flexural and tensile strengths. The water-cement ratio law seems to be borne out by these tests insofar as crushing strength is concerned. **Mr. Jackson.**

F. H. MENEFEE—With regard to the coarse aggregate, you mentioned 2½ in. as the largest. I suppose that is about the limit to which you think you should go? **Mr. Menefee.**

F. H. JACKSON—That is about the limit at the present time. **Mr. Jackson.**

F. H. MENEFEE—That satisfies me, for this reason: we use coarse aggregates because they are strong and hard and reduce the amount of cement. Also the more coarse aggregates, the less fine aggregates, and the less fine aggregates, the less cement, or rather the less voids and therefore the less cement. With less cement, we need less water, and with less water we get less shrinkage. **Mr. Menefee.**

I have read some papers recently which seem to indicate that we can use larger size aggregates. I am inclined to believe that that is going to be an error, because the coarse aggregate does not shrink. It is the fine aggregate, upon losing water, that shrinks. If the coarse aggregate is too large, shrinkage around it will reduce the tensile strength in the concrete, which will reduce the ability to resist flexure.

Industrial Concrete Floors

Wear Tests on Floor Finishes at Warehouse of R. H. Macy Co., Long Island City, New York

By John G. Ahlers, J. J. Lindon and Millard F. Bird*

The purpose of the tests described in this paper and carried out on nine types of industrial cement-floor finish was to show comparative wearing strength. All of the finishes were different; and it was hoped to determine which type of finish had sufficient resistance to wear, and still was sufficiently low in cost to warrant special consideration.

The reader is requested to refer to a paper in the 1928 *Proceedings*, American Concrete Institute, by C. E. Covell, treating of a particular type of finish recommended by him. The present tests were inspired by his paper, and the type of floor discussed by Mr. Covell appears among the nine slabs tested by the authors of this paper.

In buildings, floors are one of the most important features to an owner and in industrial structures, now so largely built of reinforced concrete, they become of primary consideration.

Only for special purposes are concrete floors covered with foreign materials and as costs often determine whether a structure is to be built or not it was thought important to the concrete industry to find if a low-cost floor finish could be produced capable of high wear resistance. Many claims for floor finish can only be considered when viewed in the light of comparative costs.

On a large warehouse designed by Robert D. Kohn, Frank H. Holden, John J. Knight, Architects Associated, the contractors obtained permission to place the nine different types of floor finish enumerated below. The authors arranged with the approval of the architects to have the contractors, the Barney-Ahlers Construction Corporation, place these slabs. The A. C. Horn Company supplied the testing machine, the owners, R. H. Macy & Company, the light current and power.

Schedule of Finishes—The accompanying diagram, Fig. 1, shows the general layout of the panels which were tested. With the exception of the two proprietary finishes all were laid at one time and therefore under the same weather conditions. They were all cured under identical conditions, namely, moist sawdust for a period of two weeks after placing. The finishes were permitted to cure thoroughly, and no tests were begun until two months after they had been placed.

* Respectively Secretary-Treasurer and Construction Superintendent, Barney-Ahlers Construction Corporation, and Vice-President A. C. Horn Sales Corporation.

Panels No. 1 and No. 3 are proprietary finishes, No. 1 having been placed by the Kalman Floor Company, Inc., and No. 3 by the American Betonac Company. In all the other panels standard materials were used, and the finishes were placed by the Barney-Ahlers company's regularly employed cement masons.

Panel No. 2 consists of a 1-in. topping composed of one part of tested portland cement and two parts of grits (no sand being used) placed monolithic with the slab. This finish was rodded off and rolled with a terrazzo roller, and troweled until it was impossible to make any further impression upon the surface with a steel tool. This type of finish was suggested by John J. Brennan, president, Local No. 570 O.P. & C.F.I.A., New York City, and was placed under his personal supervision.

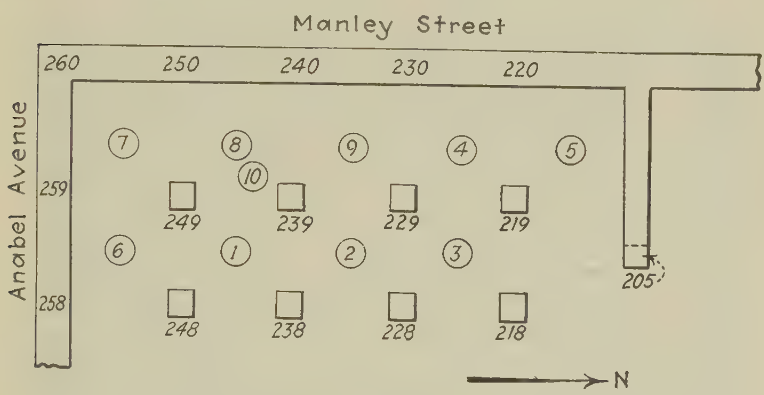

FIG. 1—FLOOR PLAN SHOWING LOCATION OF TEST PANELS, MACY WAREHOUSE.

Panel No. 4 consists of a 1-in. topping, proportion of 1 cement, 1 sand, and 1 grit, in accordance with the specifications outlined in the paper by Mr. Covell[1] referred to above, placed monolithic with the slab. This panel was rodded off, floated and given one troweling. Two weeks later the surface was ground off with a terrazzo grinder.

Panel No. 5 consists of a 1-in. topping of 1 cement, 1 sand, and 1 grit, placed monolithic with the slab, rodded off. Thirty pounds of metallic hardener per hundred square feet was floated into the surface and then troweled.[2]

Panel No. 6 consists of a 1-in. topping of 1 cement, 1 sand, and 1 grit,

[1] *Proceedings*, A. C. I., Vol. 24, 1928, p. 461.

[2] Following is the specification for metallic hardener used in these floors:

Immediately following process of leveling off the concrete slab, and after surplus water has disappeared, deposit upon the surface uniformly a dry mixture consisting of: 1 bag metallic hardener (100 lb.); 1 bag standard portland cement; and ½ bag of clean, coarse sand. This batch shall be mixed thoroughly and then evenly distributed over approximately 300 sq. ft. of surface. Work thoroughly into slab with heavy wooden floats, and finish with steel trowel. A second troweling shall be given promptly thereafter, and continued until the surface is hard and smooth. (Although a little heavy, this finish is what is commonly called a 30-lb. dustcoat.)

placed monolithic with the slab, rodded off, floated and troweled, with no hardener of any sort. (In accordance with Tentative Specifications, A.C.I., Mixture No. 2.)

Panel No. 7 consists of the slab itself without topping, poured to its full thickness and finished with 30 lb. of metallic hardener per hundred

TABLE I—PROPORTIONS AND STRENGTHS OF MIXTURES IN DIFFERENT FLOOR PANELS.

Panel	Proportions by Volume	Mixing Water	$\frac{W}{C}$	Results of 28-Day Cylinders, lb. per sq. in.	Sacks of Cement per Batch*	Weight of Aggregate per Batch, lb.
No. 9	1:5.5	Moisture: 3400 × .035 ÷ 8⅓ .... 14¼ gal. Water used .... 23½ gal. Total water .... 37¾ gal.	0.92	2300	5½	3400
No. 2	1:2	Moisture in grits: 12 × 105 × 0.26 = 32.8 lb... 4 gal. Water used .... 21 gal. Total water .... 25 gal.	0.56	5740	6	1260
No. 4	1:1:1	Moisture: Sand 6 × 93 × .057 .... 32.0 lb. Grits 6 × 99 × .026 .... 15.5 lb. 47.5 lb. = 5.7 gal. + 24 gal. = 29.7 gal.	0.66	6210	6	1152
No. 3	1:1:1 1:¾ iron	....	....	4360 base 7960 finish	... ...	....
No. 6	1:1:1	Same as No. 4	0.66	5090	6	1152
No. 7	1:5:5	Moisture: $\frac{3400 \times .035}{8.33}$ .... 14¼ gal. Water used .... 26½ gal. Total water .... 40¾ gal.	0.99	2300	5½	3400
No. 1	1:¾:2	....	....	....	...	....
No. 5	1:1:1	Same as No. 4	0.66	7000	6	1152
No. 8	1:5:5	Same as No. 7	0.99	2300	5½	3400

* CEMENT TESTS

	Fineness, Per Cent Passing No. 200 Sieve	Initial Set		Hard Set		Tensile Strength, lb. per sq. in.		Setting Time
		hr.	min.	hr.	min.	In Water, 7 Days, 1:3	28 Days	
Test No. 1..........	84.8	3	28	5	58	297	395	O.K.
Test No. 2..........	86.0	3	42	6	32	295	391	O.K.

square feet, to produce what is commercially known as the metallic monolithic surface. (See footnote page 20 for specification).

Panel No. 8 is exactly the same as No. 7, except that it is finished with 40 lb. of metallic hardener per hundred square feet.

Panel No. 9 consists of the slab itself containing a quart of calcium chloride solution per bag of cement, rodded off and finished with 30 lb. of metallic hardener per hundred square feet. (See specification bottom page 779).

A Panel No. 10 is shown in the tests made by the testing laboratories of Columbia University and given as an appendix to this paper. This panel represents a second abrasion test made on Panel No. 8.

Where the slab is spoken of this consists of a standard concrete placed with a water-cement ratio of 1.00 using cow bay sand and Long Island gravel, similar to all concrete in the structure where the average tests showed 2659 lb. at 28 days.

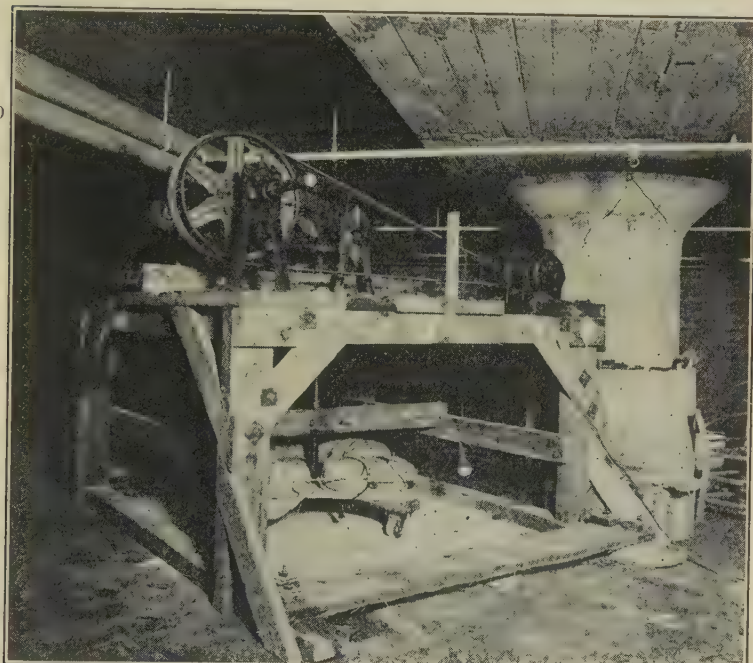

FIG. 2—ROTARY MACHINE FOR ABRASION TESTS.

Steel factory-truck wheels set to describe circles of 36, 40, 44, and 48 in.

The abrasion tests were made with the rotary machine shown in Fig. 2. It consists of a motor driven horizontal shaft geared through bevel and pinion to a vertical shaft to the lower end of which is loosely attached a square table carrying a set of four typical steel factory-truck wheels. The wheels are set to describe circles of 36 in., 40 in., 44 in., and 48 in. diameter. Upon each corner of the table is strapped a standard bag of portland cement. The table is so attached on the vertical shaft that it carries no weight from the shaft itself or any other part of the apparatus, the vertical shaft being caught in three large bearings to

guard against any side thrust. New truck wheels were used for each test of a new slab so as to give the same conditions of abrasion.

The speed of the table throughout these tests was 26 r.p.m., and as noted a new set of wheels was employed for each separate panel. In each case the machine operated for 16 hours, which we compute to be equal to approximately 10 years and 11 months of factory operation. Of course there are sections of factory floor which never receive any traffic whatever, so that the computation of any length of time must be arbitrary. Nevertheless, if we assume that under average conditions a truck wheel will pass over a certain given spot in a floor once an hour, and using 44 hours as the average factory week, then we have 26 x 60 x 16 or 24,960 hours, which equals about 10 years and 11 months.

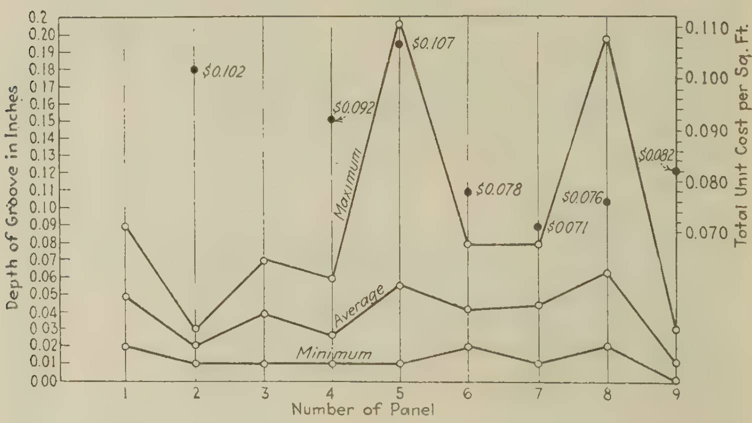

FIG. 3—DEPTH OF WORN GROOVES IN VARIOUS PANELS AND UNIT COSTS PER SQ. FT.

The depth of the grooves cut by the machine was marked in all the panels, some to a greater extent than others. The results are best learned from the appended "Report of Measurements of Wear of Various Floor Finishes," conducted by a representative of the Columbia University Testing Laboratories, and reported by W. J. Krefeld, engineer of tests. Attention is drawn to the fact that ten determinations of the depth and width of *each* of the four annular grooves in each panel were made to give a fair average and as uniform a comparison as possible.

Judging solely by the report of the Columbia University Testing Laboratories, while the various panels all showed a marked resistance to wear, some, of course, showed better than others, and we give in Table II a schedule of the tests, listed in the order of their maximum, average and minimum wear as measured in the Columbia University report. These results are also shown in Fig. 3.

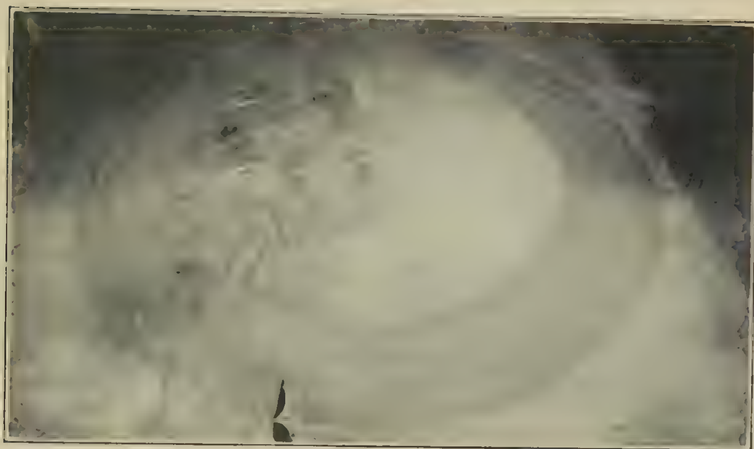

FIG 4—SLAB WITH STANDARD CEMENT FINISH AFTER TEST.
Typical of appearance of all slabs after test. Circles worn by wheels of testing machine shown in Fig. 2.

FIG. 5—SLAB WITH IRON FILINGS FINISH AFTER TEST.

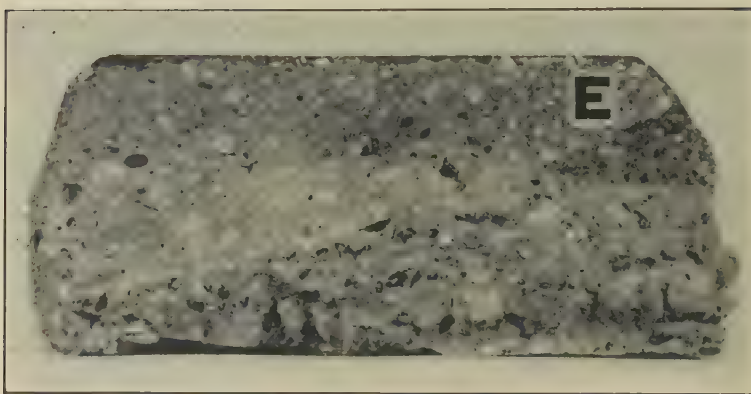

FIG. 6—SAWED SECTION THROUGH PANEL NO. 5, A SLAB WITH A 1-IN. TOPPING OF CEMENT, SAND AND GRIT PLACED MONOLITHIC WITH SLAB.

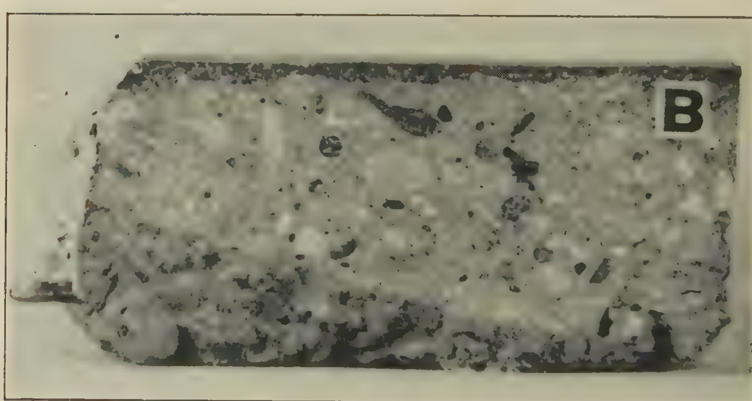

FIG. 7—SAWED SECTION OF PANEL 8, A SLAB FINISHED WITH METALLIC HARDENER.

By referring to Table II, it will be seen that the panel 9 stood up the best throughout the average, minimum and maximum ranges, followed closely by panel 2. There is a fairly large difference between panel 2 and panel 4 in both the average and the maximum, and a still greater difference between panel 4 and panel 3 in the average only.

TABLE II—DEPTHS OF WORN GROOVES IN VARIOUS PANELS (IN INCHES)

	Average	Maximum	Minimum
Panel No. 9	0.012	0.030	0.000
Panel No. 2	0.020	0.030	0.010
Panel No. 4	0.027	0.060	0.010
Panel No. 3	0.038	0.070	0.010
Panel No. 6	0.043	0.080	0.010
Panal No. 7	0.044	0.080	0.010
Panel No. 1	0.047	0.090	0.020
Panel No. 5	0.056	0.200	0.020
Panel No. 8	0.063	0.210	0.020

It is particularly interesting to note that in ten determinations on panel 9, three of the circles showed zero wear, and the fourth circle showed only 0.010 wear in the minimum range.

The panel showing the most uniform wear throughout is No. 2. The reader will recall that no sand was used in this panel, but that the topping consisted of a 1:2 mix, using grits.

TABLE III—COST DATA.

	Area, sq. ft.	Cost of Labor per Panel	Cost of Material per Panel	Total Cost per Panel	Unit Costs per sq. ft., Labor	Unit Costs per sq. ft., Material	Total Unit Costs per sq. ft.
Panel No. 9	430	$19.70	$15.40	$35.10	$0.046	$0.036	$0.082
Panel No. 2	550	31.50	24.75	56.25	0.057	0.045	0.102
Panel No. 4	430	24.00	15.70	39.70	0.056	0.036	0.092
Panel No. 3	...	.....	.....	.....	.....	.....	.....
Panel No. 6	480	20.25	17.27	37.52	0.042	0.036	0.078
Panel No. 7	390	13.10	14.23	27.63	0.034	0.037	0.071
Panel No. 1	...	.....	.....	.....	.....	.....	.....
Panel No. 5	415	22.50	21.89	44.39	0.054	0.053	0.107
Panel No. 8	430	15.05	17.21	32.26	0.036	0.040	0.076

The cost of plain concrete in the top 1 inch where no topping was used has been included in these costs.
Cost of plant and hoisting engineer not included.
Cost of grinding machine and power not included for panel No. 4.

Conclusions—The outstanding result of these tests seems to be that all the slabs gave excellent resistance to extreme abrasion and all can be said to be suitable for industrial purposes.

The table of costs has been computed with the best possible information available from such relatively small areas, but carefully weighed and painstakingly computed by people experienced in calculating costs.

It is the hope of the authors that others may carry out further tests on cheaper types of finish that might stand up nearly as well as the poorest of these. Such a finish would be of great value to the concrete industry.

Appendix—Report of Measurements of Wear on Various Floor Finishes

By W. J. Krefeld*

This report records the results of measurements of wear resulting from tests made upon various floor finishes applied to nine different panels of the basement floor of the R. H. Macy Warehouse at Manley Street and Anabel Ave., Long Island City, N. Y. The wear tests were made by J. G. Ahlers. The testing device used was intended to simulate the action of factory-truck wheels on the floor finish and in its operation the wheels were moved over the floor in circles about a central driven shaft. Four such wheels were operated at 18 in., 20 in., 22 in., and 24 in. from the central shaft resulting in the wear of annular grooves having 36 in., 40 in., 44 in., and 48 in. diameters.

At the request of Mr. Ahlers a representative of these Laboratories made measurements of the depth and width of the annular grooves resulting from the wear test.

Wear Measurements on Floor Finishes
(Average of Four Circles)

	Panel No. 1			Panel No. 2			Panel No. 3			Panel No. 4			Panel No. 5		
	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum
Depth	0.047	0.09	0.02	0.020	0.03	0.01	0.038	0.07	0.01	0.027	0.06	0.01	0.056	0.21	0.01
Width	1.59	2.12	1.00	1.53	1.95	1.20	1.67	2.15	1.00	1.60	1.90	1.30	1.61	2.35	1.00
	Panel No. 6			**Panel No. 7**			**Panel No. 8**			**Panel No. 9**			**Panel No. 10**		
	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum
Depth	0.043	0.08	0.02	0.044	0.08	0.01	0.063	0.20	0.02	0.012	0.03	0.00	0.048	0.08	0.03
Width	1.63	2.05	0.90	1.60	1.95	1.05	1.63	2.15	0.90	1.41	2.00	0.95	1.61	2.10	1.10

Record of Measurements—The location of the test panels and numerical designation assigned to each panel and referred to in the accompanying table are shown on Fig. 1. This table gives the average, maximum and minimum of measurements recorded. The measurements included ten determinations of the depth and width of each of the four annular grooves formed in each panel.

* Engineer of Tests, Testing Laboratories, Columbia University.

Discussion—Wear Tests on Industrial Floors

H. J. Gould—I was interested in reading this paper in the light of an advertisement by one of the manufacturers of iron filings who has made a similar test using square cubes and loose sand as the abrasives. He did not seem to get quite the same results that Mr. Ahlers did with the calcium chloride. I was wondering if Mr. Ahlers had made any comparison of these tests? **Mr. Gould.**

J. G. Ahlers—I am not familiar with the tests. **Mr. Ahlers.**

E. L. McFalls—I am familiar with those tests. We had them made by the Ohio State University, and the comparison was between a calcium chloride treated slab without any iron filings and a slab with the iron filings but without calcium chloride, so there was no direct comparison. **Mr. McFalls.**

H. C. McCall—I would like to ask Mr. Ahlers if he thinks that the testing machine could have been arranged so that the wheels would travel back and forth instead of in a circle, so that the wear on the wheels would have been less and the grinding action on the floor might have been less? **Mr. McCall.**

J. G. Ahlers—I believe the wear might have been less, but we were after the most possible wear. We wanted that cutting action. **Mr. Ahlers.**

H. C. McCall—The point I have in mind is that perhaps the action of the testing apparatus is different from what would result from trucking over the floor. We are considering a series of tests along similar lines at the present time, but we are considering the advisability of mounting our truck wheels so that they travel in a zigzag path, moving sideways along the specimen at the same time that the main travel is back and forth. **Mr. McCall.**

J. G. Ahlers—We believe the maximum wear occurs where trucks come around corners and our apparatus was designed to simulate that condition. **Mr. Ahlers.**

Melvin Seddeler—I would like to ask Mr. Ahlers if, in his tests, any consideration was given to the type of business which might use the floors? I am thinking particularly of floors for dairies, sugar refineries, canning factories and various other plants where you will encounter acids that are supposed to be extremely damaging to cement. For instance, in a laundry in close proximity to the washing basins the floors were worn to a greater degree than in any other portion of the same slabs, and I know that several corrective measures were used, but as yet, nothing has been applied that will stand up under the acids used in the bleaches and the washing powder. **Mr. Seddeler.**

Mr. Ahlers.

J. G. AHLERS—No, we could not do that. The number of tests that could be made is unlimited. All of us who do construction work have had our trials along this line and have had floors go bad and have had the blame put on us instead of on the materials. In our tests we only tested for resistance to wear under average business conditions.

Mr. Hart.

W. E. HART—Perhaps I can answer that, partly. In the first place, the floor should be built as dense as possible. Then it can be treated by cutting back with a volatile oil and gasoline, which is somewhat dangerous, of course. Another combination is to make a mixture of parafine, toluol and pure turpentine, mix them together in the form of a paste, and spray that on the floor and rub in with a hot iron. That will resist the action of acids about as well as anything we can get at the present time.

Standards of Performance of Concrete Masonry Units

By E. Grant Lantz*

A masonry wall has two functions—to support and to protect. With the usual limitations of wall heights due to economy and building code regulations, a 700-lb. hollow unit or a 1500-lb. brick will have a factor of safety considerably in excess of 10. Strength, however, is not the only factor to be considered in the selection of modern building materials.

The factors which provide protection are discussed in this paper, factors which are important and worthy of considerable study if the use of concrete masonry is to increase and keep pace with the records established since the war.

Temperature insulation is important and is a factor which requires further investigation after a standard laboratory method has been decided upon. The results should tell the manufacturer how to grade his aggregate to obtain the best insulation with standard strengths.

Fire-resistance is closely allied with temperature insulation. The Portland Cement Association has erected in its laboratory a furnace for making fire tests and determining, subject to final check by the Underwriters' Laboratories, the design of unit, gradation of aggregate, consistency of mix and cement content which will give to the industry the lightest and most economical unit which will pass the standard 3-hr. fire test.

Considerable difference of opinion is being expressed regarding the effect that porosity and absorption have on the resistance to weathering. Information is available but it is not sufficiently complete to offer a solution to this problem.

Sound insulation is a problem of group living but like temperature insulation a standard laboratory method should be decided upon.

While there is a definite need of more technical information on the various characteristics of concrete masonry, the equivalent of more than one million 8 x 8 x 16-in. units is being used for the construction of homes, offices, commercial and farm buildings each day on the basis of accepted merit.

Temperature Insulation—The national advertising of the insulation interests has made material buyers insulation-conscious and the concrete products industry has profited greatly thereby.

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* Secretary, A. C. I. Committee P-1—Standard Building Units. Cement Products Bureau, Portland Cement Association.

There is little test data available on sand-gravel block but it has been estimated that the heat conductivity is 90 per cent of monolithic concrete.

Because of the cellular structure cinder and Haydite units have a higher insulating value, depending of course upon the density of the unit under observation. An average coefficient of heat conductivity of 1.77 is given for cinder units and 1.62 for Haydite. This means that, for the cinder unit 1.77 b. t. u.'s will flow per hr. through 1 sq. ft. of the material when the difference in temperature is 1 deg. F. per 1-in. thickness.

This built-in insulation often makes possible a saving in construction costs through the ability to plaster the masonry wall without furring and lathing, a saving in fuel, and increased comfort for the occupants

Fire Resistance—Sand-gravel, cinder and Haydite block, 8 x 8 x 16-in. units, and Stone-tile have successfully passed the standard 3-hr. fire test of the Underwriters' Laboratories. Manufacturers who comply with the standards established for the samples originally tested may apply for the Underwriters' Inspection and Certificate Service.

This inspection guarantees to the builder that the units purchased will provide protection against passage of flame and dangerous transmission of heat for the period stated on the Underwriters' certificate, and furthermore that the units meet the physical requirements of the American Concrete Institute Specifications for Concrete Block and Tile.

No other hollow masonry unit is eligible to this inspection service.

Sound Insulation—The concentration of business and families in huge office buildings and apartmentsh as created the problem of securing sound insulation for privacy and comfortable living conditions. Acoustical properties are important in the performance of a building material in modern structures.

The noise-proofness of a partition is usually expressed in the term of "logarithm of reduction" or "reduction in sensation units." The latter is ten times the former. The C. F. Burgess Laboratories, Inc., Madison, Wisconsin, have determined from tests that a logarithm of reduction of 4.1 to 4.2 will render ordinary speech completely inaudible.

While no tests have been made on the sound absorption of sand-gravel units the logarithm of reduction will probably depend upon the coarseness of the aggregate used and the character of the exposed surface. Considerable sound energy is lost when passing through mediums of different densities, such as the two web walls and the air space between in concrete block.

Laboratory tests have shown that the logarithm of reduction for 4-in. Haydite partition tile plastered on both sides is 3.8. Assuming the values established by the C. F. Burgess Laboratories as correct, ordinary speech can be faintly heard but not understood.

The time taken for a sound to die out in a room, that is the time of reverberation, is important in the design of auditoriums. The rough surfaces and cellular structure of cinder and Haydite units when left

exposed or sprayed or painted are almost ideal for the reduction of reflected sound.

Weight—Skyscraper construction has injected the factor of weight into the consideration of building materials. The production of 8 x 8 x 16-in. units weighing 32 lb. or even less instead of 50 lb. is a considerable factor in the design of a 20-story building.

Within the past year cinder masonry units have been used in Philadelphia for four structures ranging in height from 19 to 26 stories as backing for face brick, as fireproofing for steel and in partitions.

The weight in concrete masonry is reduced by developing the cellular structure to the fullest possible degree. Strengths above 700 lb. are usually gained at the expense of lightness in weight.

Wall Efficiency—The strength of masonry can be determined if the relation between the strength of the masonry and the strength of its component parts is established.

Albin H. Beyer and William I. Krefeld report their findings of this relation in their Bulletin from Columbia University, "Comparative Tests of Clay, Sand-Lime and Concrete Brick Masonry"—

"For the concrete brick piers and the ratio R of ultimate compressive strength of pier to ultimate compressive strength of brick was found to decrease with an increase in compressive strength of brick, all other variables remaining constant. The numerical value of R expressed in terms of the compressive strength of the brick B is given by the equations:

For concrete brick, Series A

$$R = 6.2 \div B^{0.28}$$

For concrete brick, Series B and C

$$R = 5.5 \div B^{0.28}$$

"The first equation applies to concrete brick Series A for which the mortar strength averaged 2,336 lb. per sq. in. and the second one applies to concrete brick Series B and C in which the mortar strength was only 1,838 lb. per sq. in.

"For low strength brick of the A series, i.e., 1,000 lb. per sq. in., the value of R was found to average 0.89; this decreased to 0.66 for a brick having a compressive strength of 3,000 lb. per sq. in."

In a later test cinder block piers were used in determining this ratio. The average R for 2 piers approximately 8 x 24 x 54 in. built of 8 x 8 x 16-in. units and half units was 0.729. The strength of the individual units was 927 lb. per sq. in. of gross area. The R for a pier 12.40 x 24.0 x 54 in. built of 8 x 12 x 16-in. units and half units was 0.547. The strength of the individual units was 1,315 lb. per sq. in. of gross area. The decrease in R was probably due, as in concrete brick, to the increase in strength of the individual units.

Character of Bond—The high wall efficiency of concrete masonry may be in part due to the excellent bond between portland cement mortar and the individual unit. In the Columbia University tests the concrete

brick piers failed in general in diagonal shear, the resultant fractures resembling very closely the characteristic fractures obtained with monolithic concrete piers.

The report referred to under "Wall Efficiency" states that the bond between portland cement mortar and concrete bricks is as effective in producing monolithic action as the bond between mortar and aggregate in monolithic concrete piers.

This effective bond aids in the application of portland cement stucco as it acts in addition to the strong mechanical keys produced by the rough surfaces of the masonry.

Permanence—Permanence is a physical requirement that may well be placed side by side with strength. But strength is not necessarily a guarantee of permanence. For concrete masonry we should have some measure of resistance to weathering.

The University of Wisconsin found that after 100 reversals of freezing and thawing the strength of an 8 x 8 x 16-in. cinder block was 7.8 per cent greater than the strength of blocks stored in air for the same length of time. The average loss in weight of the units during the freezing test was 1.3 per cent. This test is not of sufficient length to be of value in estimating weather resistance. The theory has been advanced that the nature and distribution of the pores or cavities is an important factor in resistance to disintegration by freezing.

Prof. H. Kreuger in his paper "Investigations on Climatic Action on the Exterior of Buildings" gives some interesting data on porosity and the power to absorb water as factors of importance in determining the resistance of brick work and plaster to freezing. Since most of the investigation was confined to these two materials it has not been established that the theories and data from the following paragraphs apply to concrete.

"The property of resisting frost depends very largely on the ratio between the power to absorb water and the porosity. It is obvious that where the power to absorb water equals porosity, in the case of a surface material which has absorbed water to the extent that all pores are filled, exposure to frost accompanied by a 10-per-cent increase in the volume of water must inevitably lead to disintegration of such material unless there is some way for the water to escape. It is possible to fill all pores with water under high pressure and it has been shown that even material of extraordinary strength can be frozen to pieces.

"If the pores have only been filled with water to the extent of 90 per cent there is theoretically sufficient space for the water to expand in the pores when it freezes to ice. The distribution of water in the interior of a material will, however, never be so uniform that the surplus cavities are situated in the right places, and for this reason it generally happens that a stone having $w/p = 0.90$ becomes fractured when w denotes the power to absorb water and p the porosity. If the coefficient of water absorption a is denoted by w/p experiments prove that when a is greater than 0.85 the material is generally destroyed in the freezing test, whereas a coefficient of water absorption gelow 0.80 generally indi-

cates a material that is well able to resist frost. It does, however, happen that one part of a stone has a higher coefficient of water absorption than another, and attention should be directed to possible inequalities in the structure."

Summary--This resume of the available data on concrete masonry emphasizes those properties other than strength that have aided in the general acceptance of this material. Strength is important, but hardly overshadows temperature insulation, fire resistance, sound insulation, weight, wall efficiency, bonding power and permanence.

Standards of Performance of Concrete for Stucco

By W. D. M. Allan*

Portland cement stucco can be defined as a portland cement mortar applied in a plastic state to exposed walls or surfaces. Usually stucco is used for decorative purposes, but frequently only for protection against elements. With few exceptions, protection is of first consideration. Assuming this to be true, the necessary properties of durable stucco are easily found.

Workmanship and adherence to established fundamentals of good concrete practice are necessary in all concrete products, but probably nowhere in the entire concrete field are they more important than in stucco. The discussion in this paper is limited to the requirements for portland cement stucco materials and does not touch on workmanship or construction details. At the same time, I recognize that methods of construction and application are just as important to a satisfactory stucco job as quality of materials. No amount of high grade materials will take the place of intelligent workmanship and proper construction methods. Nearly every one of the properties listed and discussed later in the paper are affected to a marked extent by the method of application and the kind of a backing over which the stucco is applied. For the purpose of the paper, however, it is assumed that high grade construction methods are followed and the discussion limited to the requirements of the materials, which in the hands of intelligent workmen will produce the results properly expected of portland cement stucco.

Portland cement stucco is a concrete product having all the properties of concrete, and, if considered as such, much of the known data regarding concrete will be of service in helping us to understand how to obtain the best results from stucco. Referring to our definition of portland cement stucco, we can list its standards of performance as follows:

Durability
Workability
Watertightness
Resistance to frost action
Freedom from discoloration
Freedom from crazing
Windproofness
Fire resistance
Ability to protect reinforcing metal

* Manager Cement Products Bureau, Portland Cement Association—Chairman, Sub-Committee on Stucco, A. C. I. Committee C-3.

These properties could be subdivided, but for the purpose of a brief summary they should suffice. No attempt has been made to list them in the order of their importance. This could hardly be done for the country as a whole because no one arrangement would hold for all sections. For example, standards of durability are much more rigid and, for that reason, more important along the New England seaboard than in southern California; watertightness in Arizona is not nearly as important as watertightness in Florida; frost action certainly is more important in Minnesota than in the southern states.

Durability, or permanence, is doubtless a property builders expect of portland cement stucco. While there are no definite standards available for measuring the durability of stucco, experience has taught builders what they can expect in the way of long-time service from portland cement mortar in thin, concrete slabs. Not all stucco materials are durable. Some fail because they are unstable chemically, others because their use on exteriors forces the material beyond its natural limitations.

Durability is probably the broadest of the properties listed and overlaps with some of the others, but for the most part is a distinct and important requirement that builders set up for the product. Because of the recent general failure of an exterior stuccoing material, not made of portland cement, builders in general, at the present time, attach much more importance to durability in stucco than they have in times past. Later in the paper the discussion of other standards of performance for stucco will throw some light on ways for obtaining durability.

Since we are considering portland cement stucco as portland cement concrete, all of the factors that tend to make durable concrete in pavement slabs, bridge piers, dams, or concrete structures exposed to alkali conditions, have a bearing on durability in stucco and the stucco manufacturer should be familiar with the research work that has been done in these other fields of concrete in order to have a proper appreciation of the fundamentals of good concrete that must be worked into his stucco material.

Our investigations indicate that the properties of stucco that tend toward durability are affected considerably by the amount of mixing water required to make the stucco workable. That combination of aggregates that will give the desired workability with the least amount of mixing water will tend to give the highest resistance to destructive weathering agents. More study is needed on this phase of stucco, but strong tendencies have already been developed.

Workability is a more rigid requirement in stucco than in most forms of concrete. A considerable range in workability is permissible in a concrete slab or a reinforced concrete wall because additional spading will usually overcome a considerable range in workability in these uses of concrete, but in stucco the material must hang to the workman's trowel in a very definite manner which, in turn, fixes the degree of workability within narrow limits.

Several attempts have been made to use the flow table as a measure

of workability, but in general they have not been satisfactory. Plasticity measuring devices have been used by some investigators, and these are satisfactory for stuccoes that are very nearly alike. However, when the composition of the stucco is changed slightly, such as in the grading of the aggregate, the nature of the admixture or proportions of aggregate to cement, any of the plasticity measuring devices that have come to my attention are not satisfactory.

In our investigations, a variation of one per cent in the amount of mixing water required has, in many cases, meant the difference between a mortar that is too dry or one that is too wet and rarely will the difference be more than two per cent between the wet and dry limitations except where large quantities of asbestos fiber or asbestos flour are used. The most satisfactory measure of workability at present is the application of stucco to a base coat panel.

Within the last 4 or 5 years, workability in finish coat stucco has taken on an entirely new meaning. Few, if any, of the textures used in portland cement stucco today are new. Many of them are thousands of years old but are relatively new for application in portland cement mortar. The plastering materials that these textures were worked out in originally were generally much fatter than portland cement stucco and a higher degree of workability was easily obtained with these materials. However, they did not have the elements of durability and permanence that are required for the general use of stucco in all climates. As a result, a great deal of attention has geen given during the last few years to methods of obtaining a degree of workability necessary to develop these textures without sacrificing strength and density and other properties associated with durable stucco construction under rigorous weather conditions.

Workability is generally obtained through the use of plasticising agents, very fine admixtures used for fattening the mortar. The use of plasticising agents at the expense of proper grading of aggregates is usually followed by an appreciable reduction in strength, increase of absorption and lack of resistance to weather. In general, the best stucco is obtained through the use of that grading of aggregate which requires the minimum amount of plasticising agents in order to give required workability. At this point, I wish to make clear that there has been a marked tendency to overstress fatness in portland cement stucco. There has been a feeling that portland cement stucco can have all the fat, buttery slip under the trowel that is obtained from some other plastering materials without sacrificing durability. While sufficient workability can be obtained in portland cement stucco for the application of any kind of textures, the plasterer must recognize that the material has to be applied in a different manner from that in which lime, gypsum or magnesite plasters are applied.

Watertightness in any protective covering for a building is taken as a matter of course. This term is used in a restricted sense, realizing that absolute watertightness can only be obtained at very great cost and by

methods not practicable in building construction. Portland cement stucco must have low capillary attraction in order to be sufficiently watertight, otherwise water entering the wall may injure the backing and render the structure unsound.

Watertightness, to a high degree, is obtained in portland cement stucco by proper grading of aggregates and adherence to the other fundamentals of good concrete, such as thorough mixing and proper curing. In the proposed specification of Sub-Committee 1 on Stucco of Committee C-3, measure of watertightness is established through an absorption test. This test requires that the specimens shall be cured wet for 6 days and then allowed to stand for 21 days in the dry air of the laboratory, after which they are immersed in water for 24 hours and the percentage of water absorbed measured. There are strong indications that the rate of absorption of water in stucco is more important than the ultimate absorption, but since no standards have been set up for measuring the rate of absorption and since it seems apparent that several committees in the American Concrete Institute and the American Society for Testing Materials and other technical organizations will probably give considerable thought to the matter of rate of absorption, the Stucco committee felt that the present procedure is the best one available at the present time.

Resistance to Frost Action in most sections of the country is an important requirement for any exposed building material. It is particularly important in stucco because where it is used for decorative purposes the fine textures developed must resist frost action or the textures will be lost. Resistance to frost action is measured in the laboratory through freezing and thawing tests. Some investigators claim that four cycles in the refrigerator are equivalent to the destructive action of one winter in most sections of the United States.

The exact relation between composition and internal structure of stucco and resistance to frost action is not definitely known. Methods of procedure for conducting freezing and thawing tests have not been standardized. For this reason it is difficult to compare freezing and thawing results obtained by different laboratories. Considerable research is required to standardize methods of conducting freezing and thawing tests, and to interpret the results in terms of resistance to weather. In view of the fact that freezing and thawing tests are expensive in that they require a long time for completion, it will be highly desirable to establish the relation between rate of absorption or total absorption, that can be conducted in a short time at relatively low cost, and resistance to freezing and thawing. Several investigators are giving thought to this problem at present.

Freedom from Discoloration is especially important where stucco is used for decorative purposes. There are no definite standards of performance established for this property and yet it is one that architects and builders consider of prime importance.

Discoloration generally results from efflorescence, fading or staining.

Efflorescence is the most difficult form of discoloration to control, in that weather conditions play an important part. In general, efflorescence is caused by the same process in all building materials where it appears. Very few, if any, masonary or plastering materials are free from efflorescence in some form or degree. Efflorescence is formed by water contained in the wall, or entering the wall, dissolving soluble materials and carrying them through the capillaries to the surface, where the water is evaporated and the salts deposited. If the capillaries in the wall are of such a nature that the water can enter it easily, so that there is an alternate wetting and drying action, the deposit of salt, or efflorescence, increases in thickness and becomes very objectionable, hiding the color under it and causing the wall to have a faded, blotchy appearance. Stucco applied during a long wet period is likely to show more efflorescence than that applied in a dry period. This is true also of masonry walls. The amount of water that enters the wall is greater, consequently the amount of soluble material deposited on the surface is increased.

Barring the factors, such as weather, that cannot be controlled, the stucco that becomes plastic with the least amount of mixing water will have the least tendency to effloresce, because, in general, it is denser and has fewer capillaries. The use of soluble admixtures in stucco frequently leads to a relatively large amount of efflorescence. Proper grading of aggregates to produce very dense mortar is probably the best guarantee against objectionable efflorescence.

Fading is generally caused by inferior pigments or improper mixing. Guaranteed lime and sun-proof mineral oxides should be used for color.

Staining results from dirty materials or porous mortar that permits water to leach out substances from the backing that, when brought to the surface, cause discoloration. Dense mortar, as measured by the absorption test, properly applied and cured is the best safeguard against staining.

Freedom from Crazing is important largely because of the psychological effect on the purchaser. Crazing is usually not structurally dangerous and should be, for the most part, overlooked. However, manufacturers of stucco should take those precautions necessary to limit to a minimum the tendency to craze.

Crazing, like efflorescence, is caused as much by weather conditions over which we have no control as by the material. It is, in general, those stuccoes that have the largest percentage of fines in them that tend to craze most readily. In the proposed specification of Committee C-3, a provision has been included to guard against crazing, in that not more than 35 per cent of the total weight of the dry stucco sample is permitted to pass a 100-mesh sieve. This requirement makes it necessary for the manufacturer to use a well-graded aggregate in his stucco, so that he can obtain his workability with the least amount of plasticising agent and, in turn, with the least amount of mixing water.

Windproofness and Fire Resistance are recognized properties of portland cement stucco and have considerable value in reducing heat losses and increasing periods of fire resistance. Three-quarters of an inch of

portland cement plaster on exposed concrete masonry and clay tile walls increase the fire resistance period of the 8-in. wall by approximately one-half hour. This fact was developed by the Underwriters' Laboratories and the Bureau of Standards fire tests.

Ability to Protect Reinforcing Metal against corrosion is very necessary because whenever portland cement stucco is applied over any but sound masonry backing it should be applied to metal reinforcement. Experience with thin reinforced concrete slabs shows that dense mortars are required. In stucco as in concrete this means low water-ratio stucco. The proposed absorption test will act to produce dense mortars.

In this discussion of standards of performance of stucco, I have attempted to outline the principal properties and requirements for the material, without particular regard to strength. Strength, however, is to a considerable extent a reliable way of measuring other desirable properties in portland cement stucco and in the specification proposed by Committee C-3 for finish coat stucco material, a strength of 2000 lb. per sq. in. is required when the stucco material is tested as 2-in. cubes at 28 days of age.

In closing, I wish to emphasize that probably nowhere in the field of portland cement concrete is the importance of workmanship and con struction details of greater importance than in stucco.

Discussion—Performance of Concrete for Stucco

Mr. Pearson.

J. C. Pearson—I have had the pleasure of serving with Mr. Allan on this committee and I have subscribed to the specification, at the same time realizing that these things do not mean a great deal in reference to the quality or the properties of the stucco that we want to get. Think of requiring 2000 lb. per sq. in. for wall covering! The 2000 lb. can only be regarded as an index of durability which we think is indicated perhaps as well, at this present time, by strength as anything else.

We have a specification for gradation which says that not more than 35 per cent shall pass the 100-mesh sieve. The only purpose of this is the hope to get a decent amount of aggregate into the stucco mixture. Finally, in regard to the absorption I got into trouble yesterday because I did not think absorption meant anything at all. The point is that the 2000 lb. and the 10 per cent absorption have nothing to do with these same values when the material is on the wall and applied on an absorptive base. The biggest enemy portland cement stucco has is its tendency to shrink. I would be willing to sacrifice three-quarters of that strength value if I knew how to make a stucco that would not shrink. Unfortunately, as Professor Davis showed in his paper, those shrinkage values come in a great measure through the influence of the base, so I do not feel that we are at the end.

Mr. Spayth.

F. J. Spayth—In stucco, as in any plaster operations, the shrinkage is the item that is given the biggest consideration. It is shrinkage which causes crazing in the interior plaster operations which have been carried on over numerous thousands of years, and for this a solution has been worked out. Some progress has been made in exterior work. I think the grading of the aggregates will bring possibly greater results than anything else. There is also the question of lime which is required by U. S. government standard specifications and others. None of these take the quality of that lime into consideration. There is considerable difference in the shrinkage of two limes. Therefore I say that if the committee does take into consideration the admixtures, that they should consider not only the quantity that is used but also the quality.

Livability of Concrete Dwellings

By S. C. Hollister*

The Problem—The first requirement in the construction of any dwelling is livability. It must be free from dampness from beneath, from rain or snow-water from above, and from the elements driven against the sides. It must be capable of being easily heated during winter, and must offer protection from excessive heat in summer. In short, it must afford shelter and comfort for its occupants.

Concrete, because of its plastic nature, is admirable for building construction. It has not as yet been used extensively in the monolithic construction of small dwellings, although its use for large apartment buildings and hotels is quite general. Its trial in dwelling construction has proved its fitness as an economical material, capable of meeting any desired architectural form. Unfortunately, however, the first essential of dwelling construction—livability—was not in most cases properly developed in monolithic concrete structures, with the result that an unwarranted prejudice has been lodged against the use of concrete for dwelling purposes.

Monolithic concrete houses have been built with proper foundations, adequately strong walls and floors, and roofs that would not leak. From the standpoint of strength and rigidity, they have been wholly satisfactory. But, in most instances, in the northern localities at least, the occupants have complained of damp walls and ceilings, and cold, clammy walls and floors. In not a few instances, colds and rheumatism were charged to be the result of this unlivable condition.

Whence comes this dampness and clamminess, noted both on first and second floors? The foundation walls were of sound, well-placed concrete, and the first floor elevated above ground level or built over a basement. The walls were 4 in. or more in thickness of good quality of concrete, furred before plastering on the outer walls, and stuccoed on the exterior. Reinforced concrete floor construction was cast to be integral with the walls, and usually was plastered on the under face to form the finished ceiling. Roof construction was usually flat, as well-built as the floors, and covered on top with a standard slag-and-felt covering. Floor surfaces in some cases were linoleum cemented to the concrete slab, and in others, wood flooring on wood sleepers. Why, then, should ceilings be cold, and not infrequently sweating, and even inner walls of concrete clammy and cold to the touch?

* Consulting Engineer, Philadelphia.

Comparison of the condition of many of these houses with dwellings of various forms of masonry show the latter to be free from many of the undesirable characteristics that render the monolithic concrete houses unliveable. One obvious difference is the wood joist constructions common to the masonry houses. Another is the frame construction of interior walls. Roofs on masonry houses are either pitched, thus forming an attic, or separated from the upper ceiling by an air space of a foot or more, if laid flat, whereas in the concrete house, the roof slab commonly forms the upper ceiling as well. The masonry walls, laid in lime mortar usually, are seldom cold unless plastered directly upon, in which case they are nearly always clammy.

The structural superiority of the concrete house not only as regards structural integrity but complete fire resistance, and its economic possibilities make desirable the study of the causes of unlivability and their complete removal, before such construction can take its rightful popular place.

Causes of Unlivability—Excepting the obvious attacks of the elements, the chief source of unlivability is coldness of walls, and clamminess of walls, ceilings or floors. The coldness and apparent dampness are inter-related and frequently interdependent. A consideration of other forms of construction will shed much light on the source of both.

An ancient method of building walls that in a primitive way resembles modern cast concrete is *cob* construction, consisting of clay with intermingled straw as a binder. Houses now standing in England have been occupied continuously for over 400 years. The walls are usually from 2½ to 5 ft. thick and are plastered inside and out by applying repeated layers of lime whitewash. A foundation of masonry, usually brick, is first laid down, beginning below frost line and extending to about a foot above ground. A liberal application of hot coal tar is then given the upper face of the masonry. A layer of stiff mud made of a mixture of sandy loam and clay is next applied, with straw scattered through it as applied. This is compacted in place by tramping with the feet. No forms are used. After the wall is thus built up to a height of about a foot, it is allowed to stand in the sun for a week or more to bake. Another layer is then applied as before, baked, and so on, until the wall is completed. Both faces are scraped smooth to be plastered. The building of a two-story wall usually requires an entire building season. Obviously, in this method of construction it is necessary to prevent dampness from entering the wall because of consequent disintegration; and this is accomplished, first, by the tar seal over the foundation masonry, and secondly, by the hard shell of plaster on both faces of the wall.

Another method of construction, called *pise*, consists of earth rammed while damp into moulds to form bricks or blocks, or rammed into forms for walls. It was in use in Pliny's day and is still employed in France and England. Protection from dampness is given *pise* in the same manner as for *cob*, namely, by an under-seal of tar and plastering on both faces.

In this country *adobe* construction much resembles *pise* in method. Blocks of *adobe* are sun-baked before laying and are sometimes burnt. Because of the dry climate where this construction is common, there is seldom any dampness to combat and an under-seal is not used.

Examination of old walls of brick or stone laid in lime mortar show marked deterioration of the mortar due to dampness from below, unless an under-seal has been used. This deterioration is noticeable 6 ft. or more above grade.

The above citations are examples of the rise of dampness from the subsoil, if the wall possesses the power of absorption and capillarity to any degree. Even stone and brick possess some capillarity and absorption, although the capillarity is often reduced by voids in the layers of mortar. Concrete possesses absorption and because of the continuity of monolithic walls, possesses capillarity to a marked degree, especially when the moisture thus drawn up from the ground is evaporated off the face of the wall at the upper levels.

The thermal properties of concrete at ordinary temperatures are well established. Stone concrete is a good conductor of heat and is not as good a material as brick or stone for thermal resistance. Furthermore, the presence of moisture in the pores greatly increases the conductivity. Obviously then, concrete walls need more attention to reduction of capillarity and thermal conductivity.

Untreated, lean stuccoes are slightly absorptive, depending upon the mix. Evidence of this is found in the behavior of such stucco upon various bases under alternate wetting and drying; also, frost action reveals the penetration of moisture into stucco, especially if it is laid against an impervious base.

This absorption beyond the surface of concrete or stucco is due to the capillary flow of moisture through the mass. This flow is stimulated by lessening the capillary gradient within the mass; and the lessening of the gradient is readily accomplished if evaporation occurs on the side of the mass opposite the absorbing surface. Moisture thus is taken up on the absorbing surface, floors, or is pulled by capillary action, and evaporated on the opposite face. When such action occurs, the thermal resistance is lessened by two or three times.

Instances have been found where incipient moisture travelled 10 ft. or more through concrete before being evaporated. Other instances were found of cold walls with moisture coming from below, and because the faces of the walls were treated, the moisture was not evaporated, but was flowing parallel to the faces.

Features of Construction Necessary to Livability—The features requiring special attention in concrete house construction in order to secure livability are seen to be:

(1) Prevention of moisture rising from foundations.

(2) Prevention of moisture absorption

- (*a*) through the walls
- (*b*) from walls into floors and roof slabs.

The circulation of moisture upward from the foundation into the walls may be prevented by sealing the top of the foundation wall at a level not less than a foot above the ground level with a generous application of a good bituminous compound, a practice which should be followed in brick and stone construction also, to prevent deterioration of mortar joints. Circulation can be reduced or even prevented by choosing aggregates and mixtures that will reduce capillarity by increasing porosity. Cinders are excellent for such a purpose, besides possessing many other qualities desirable in residential wall construction.

Instead of being an undesirable feature, porosity is quite a desirable one for wall construction. Such a wall, besides possessing only a negligible capillarity, offers high thermal resistance and provides good mechanical bonding for stucco. Moisture will not pass through such a wall if a reasonably good stucco has been applied to the exterior. For ordinary residential construction a compressive strength of 800 lb. per sq. in. in 28 days is ample, when thicknesses of wall comply with rules for thickness of brick, block or tile walls of similar length and height.

In climates where there is prolonged temperature below the freezing point, and especially where the predominant precipitation falls during the cooler months, the exterior walls of dwellings should be furred before lathing, unless the walls are of porous material in excess of 8 in. in thickness. These restrictions may be modified in more moderate temperatures and under conditions of low humidity during freezing weather.

The junction of floor or roof slabs with the exterior walls must receive special consideration. The edge of the slab must extend into the wall a distance necessary to obtain proper bearing, but it should not extend *through* the wall. At least 3 in. of porous cinder concrete between the edge of the slab and the outer face of the wall is desirable; and in addition, the portion of the slab built into the wall should be well coated with a good bituminous compound.

Experience with a considerable variety of construction has taught the writer that if the foregoing precautions are adhered to, the resulting dwellings will be as livable as those of any other construction, and in addition may possess the qualities of economy and fireproofness possible with all-concrete construction.

Discussion—Livability of Concrete Dwellings

R. M. Thompson*—Mr. Hollister's paper has been read by the writer with great interest. The writer had a similar problem in Western Canada, where the temperature falls on some days to 60 deg. F. below zero, and the average temperature for 5 months would be about zero. In addition to these low temperatures, strong winds are blowing almost constantly and calm days are the exception. Mr. Thompson.

Experiments were begun about 15 years ago with the intention of designing a fireproof house as cheaply as a frame house. As a charter member of Saskatchewan Architectural Association, the writer persuaded the Association to petition the Saskatchewan Government to build a number of experimental houses for the purpose of determining the insulating value of various styles of construction. The Government agreed to the proposal, and turned the work over to the University of Saskatchewan, where a committee was formed consisting of Professor Greig, Dean McKenzie, Professor McGougan, and the writer.

About nine houses were constructed, the inside measurements of which were 6 x 6 x 6 ft. The benefits of these experiments were invaluable and the data collected were of great assistance in the construction work which was going on in the province.

The writer has not yet built a small dwelling house, although in all he has designed and built buildings in his fireproof construction to the value of $203,000 as follows:

	Cost†	Cost Per Cu. Ft.
Lanigan School (180 pupils)	$24,500.00	28.5¢
Kinistino School (270 pupils)	32,364.00	24.2¢
Y. W. C. A. (26 bedrooms)	41,503.00	32.7¢
Unity Hospital (25 beds)	42,000.00	29.3¢
Nurses' Home	24,000.00	....
Dairy Factory	15,000.00	....
Power House and Laundry	24,000.00	....

† The costs are for completed buildings and cover all mechanical trades.

All these buildings were two stories and basement, and in one or two cases provision was made for a future story. The writer endeavored to standardize all walls and pilasters, with the result that all pilasters were made 12 x 12 in., and all walls 5 in. thick, in both cases the dimensions holding constant from basement to top. Incidentally walls and pilasters were poured at the same time.

* Structural Engineer, Smith, Hinchman and Grylls, Detroit.

Three methods were used: ordinary concrete forms, shutters and sliding forms. The floors were solid slabs and beams. The interior floors were usually finished with flooring nailed to sleepers attached to the concrete with galvanized iron strips. No cinder fill was used. On one or two occasions linoleum was utilized. The interior walls usually were finished with one coat of finish plaster on concrete, but sometimes a two coat finish was used.

The extreme cold in winter and great heat in summer (at one time 104 deg. F.) made it necessary that some form of insulation be used. Over the whole basement, before the concrete was poured, two layers of building felt well mopped with pitch were laid on 4 inches of ashes. A similar procedure was used on the exterior of the outside walls up to grade. Above the grade line and on roof, the whole of the exterior of the building was covered with a special insulation material, and on top of this stucco on metal lath or chicken wire was used. The writer never has had any complaint regarding cold or clammy walls in these structures.

Regarding the walls, two experiments, one of which was involuntary, may be of interest. The janitor of the Kinistino School broke one of the sections of the boiler, with the result there was no heat in the buildin from 8 A. M. to 4 P. M. The classes were not disturbed in any way in spite of the fact that the temperature outside was 10 deg. F. below zero and the wind was blowing at 10 m. p. h. The principal informed the writer that he had visited each room hourly to observe the thermometers and that only two of them had fallen in the room on the windward side of the building, and these had dropped from 65 to 63 deg. F.

Perhaps some of you have tried the old trick of placing a brick in the furnace, then wrapping it in an old blanket and using it as a footwarmer for a sleigh ride. The writer, on thinking over the matter of the school, came to the conclusion that the walls acted like the hot brick, inasmuch as the concrete walls had been heated before the boiler broke and the heat, being held in the building by the insulation, kept the rooms at an even temperature. If this theory is correct, concrete buildings built with this type of construction should be ideal for the building of houses.

The second experiment was made in the Lanigan school. The principal wrote me, stating that he had been making temperature tests at the floor level and 6 ft. above the floor. To his surprise, he found the difference in temperature was only 2 deg. F. In his former school of hollow tile and brick it had been over 40 deg. F. Even in a well-built frame house with the wind blowing at 18 m. p. h. and temperature outside 16 deg. F. below zero the difference of temperature between the floor and the 6-ft. height runs about 18 deg. F. The writer has been assured by the principal of the Lanigan School that the building is very cool in summer.

Provision for storm sashes has been made in all these buildings, but up to date they have not been installed. The writer considers the results achieved of sufficient merit to induce him to take out two patents on his method of construction.

Standards of Performance of Concrete for Reinforced Concrete Buildings

By Nelson L. Doe*

Throughout the building industry the use of reinforced concrete is universal and constantly increasing. Its flexibility as a structural material is one of the main reasons for this widespread development. This very flexibility, however, which permits concrete to be used for such a great variety of structures, burdens the builder with a certain amount of responsibility in the design and control of the concrete for use under different conditions. Buildings, as a class of construction, present many unusual situations which should be given consideration when the specifications for the concrete are written. Each individual building may, or may not, present special problems. It is, therefore, impractical to make a specific statement giving the requirements of concrete for universal building use. Only the general characteristics can be given for the field as a whole; the varying importance of the different qualities of the concrete being determined by the requirements of each particular structure.

If it were possible to select a single quality and designate it as the most important quality of concrete for reinforced concrete buildings, the one selected would without question be *uniformity*. The structural concrete for a building must possess uniformity if the finished product meets the intended requirements. A design need not call for concrete of high compressive strength, but each member of the structure must meet the requirements of the design or the factor of safety will be a varying quantity with the different members. On building work, it is especially important that the reinforced columns which carry the loads of the stories above, possess uniformity, as on these columns the safety of the whole structure depends. Floor slabs and beams also must possess uniformity, as any portion of a building is liable to receive the full load for which it was designed, or in many cases heavier loads. Usually these members are of comparatively small section, seldom being more than twelve inches in thickness.

Considering this fact, the importance of uniformity for each batch of concrete placed in them is obvious at a glance. With the exception of some special classes of reinforced concrete products, such as precast piles, bridge arches, etc., it is quite probable that building work demands more uniform concrete than most of the other types of construction. With

* General Superintendent, Turner Construction Co., New York City.

this in mind then, that concrete must be a uniform product to meet the requirements of reinforced buildings, every effort should be made to achieve this result, not only in the selection of materials, and their proportioning by specification requirements, but by a careful study of plant layout, general field conditions and the results obtained from repeated tests made under actual working conditions.

Inasmuch as actual field tests of compressive strength are the most logical and satisfying assurance that the concrete possesses uniformity, they are especially desirable and should be made frequently on large building operations. To those in the building industry who are already interested in producing uniform concrete, regular tests are of such value that many individual testing laboratories have been established. Contractors have found that the knowledge obtained by making these regular tests has more than repaid for the cost of testing and on many operations where the specifications do not require field testings, the concrete is tested, and a careful summary of all tests recorded. To those who have not yet realized fully the modern trend of concrete control, regular tests will be a revelation.

Another very important quality which concrete must have to be suitable for buildings is *workability*,—a rather indefinite feature to define or regulate theoretically, but one which must be obtained in the field on all high class work.

Concrete buildings are as a rule heavily reinforced, with members small in section, when compared to other works of concrete, such as dams, retaining walls, foundations, etc. In order to be effective, the reinforcement of such members must be completely enveloped by the concrete of the structure. This means that proper aggregates must be selected of such size and grading that the concrete can flow in the spaces between the reinforcing bars, and that the workability of the concrete must be such that it will move along as a mass instead of separating into its different elements.

The workability of the concrete is not an item that affects the strength of the structure alone. It is a large factor in the ultimate cost of the concrete. A well designed mix, producing a workable mass, requires much less labor to place in the forms, flows more easily around the reinforcement, and shows less honeycombing or leakage through open seams, than concrete with little workability. Furthermore, the pointing or finishing of concrete surfaces in general costs much less where workable concrete has been used than where a harsh mix has been permitted. Workability is a subject which must be controlled almost entirely in the field and with uniform aggregates once combined in proper proportions, it can readily be obtained. The cost of obtaining workability can be varied to a considerable extent, however, by a variation in the kind or quantities of the different aggregates, and in each market an earnest effort should be made to select that combination of aggregates which will produce the most workable concrete at a minimum cost. These facts should be kept in mind when specifications for concrete are being written, and

considerable latitude left to the field engineers or contractors, for selection of materials to meet the requirements of the proposed structure. If this selection of materials is wisely made, no strength will be sacrificed but a workable and economical concrete will be produced.

The appearance of concrete surfaces, both on the interior and exterior of buildings, is largely a measure of the workability of the concrete itself. Concrete used on reinforced building work has too long been considered simply as a structural material, the only requirement of which was strength. Concrete buildings demand more than that. The surfaces of the concrete structure is often the finished surface of the building on both the exterior and interior. Building concrete is subject to wear and weather and must be properly made to stand up and give service, under these conditions, for a period of years.

One of the greatest causes for deterioration of exterior concrete surfaces is the placing of the reinforcing too close to the exterior form. Especial attention to this detail always is important, both in the design of the structure and the field execution of the work. This "rusting out" of the reinforcing is greatly increased if the concrete surrounding it is of a porous nature or imperfectly placed, as the dampness penetrates to bars which are close to the surface, although not actually exposed, and the rust formed soon spalls off the concrete. Properly designed and workable concrete is much less porous, and easier to work in place, hence less trouble of this nature results.

Our ability to control the strengths of concrete mixes has increased rapidly during the past few years. With the increasing number of strength specifications appearing on building work, the importance of knowing the proper mixes and varying qualities of the aggregates of any locality is evident. Often on building work, concrete of different strengths is called for. It is imperative that the necessary changes in the proportions be made correctly and with assurance that proper strengths will result. This point again only illustrates the need of a steady checking of design, etc., of mix aggregates, by repeated field tests daily. In the past, perhaps through blind supervision, or control, many yards of concrete have been placed with little regard to economy. Since the control of strength has been improved, however, much closer results are being obtained and eventually considerable economy should result. This brings up another feature of concrete for building work—the subject of economy. The vast bulk of the building industry, especially the concrete field, is competitive. Economical detailing of structural members and proper building designs that will save the owner money are continually sought after. In like manner, the economical design of concrete mixes is worthy of study and when accomplished is a distinct benefit. To save money by increasing the yield obtained from a barrel of cement means the giving to an owner more building area per dollar—and that feature should not be overlooked when concrete specifications are being written, or when the concrete aggregates are purchased, or when the concrete plant is set up.

Conditions must be studied and definite knowledge of the different factors entering into the cost of concrete obtained, if true economy is to be the result.

Those engineers and contractors who possess such knowledge and realize the importance of properly designing concrete for economy, are certainly better able to serve the industry well, and to give value for money received than are those who make little study of aggregates, or keep no record of concrete tests.

Briefly to summarize then, the standards of performance required of concrete for building work, in their order of importance, are (1) uniformity; (2) workability; (3) control of strength; and (4) economy.

Discussion—Concrete for Buildings

Clyde E. Locke—Pertaining to economy and workability as applied to reinforced concrete buildings, I wish to bring up something in connection with the water-cement ratio which is a little different from what has been previously discussed where the economy is based on a saving of cement. I would illustrate this by an example consisting of a 6-story reinforced concrete warehouse building, designed for a 400-lb. floor load. The mix used was 7 bags per cu. yd. The strength obtained was taken advantage of in the design by a reduction in the sizes of slabs, beams, girders, etc. The result of this was a saving in the amount of forms, a somewhat additional cost in steel, considerable less finishing of ceiling, quicker stripping enabling the contractor to make a speed of approximately six floors in six weeks. The finish of the floors also was worked out very cheaply. I think this is in line with what Mr. Ahlers was suggesting this morning, that some work be done in the way of developing cheaper floor finishes. In this case a beautiful looking floor was not required. The amount of water was such that, after screeding off the floor, there was no indication of additional water at the top of 30-in. beams; and yet the mix was so workable, being so rich, that it flowed around the steel with very slight labor. The finishing was done in the following manner: at the proper time the top was worked with a wood float and trowel, both being used simultaneously by the same finisher, and then, after the proper period, another troweling was given which was done very quickly and with very little effort. This was done with absolutely no dusting of any sand and cement, as there was no water to dry up and there was nothing to peel off afterwards. The cost of this was not over two cents per square foot for labor, and, as you can see, nothing for material. The aggregate used was a mixture of sand and gravel, which is practically standard in the Buffalo district. **Mr. Locke.**

A. W. Munsell—In connection with the economy feature the last gentleman mentioned: In the Hudson River bridge on January 20 we had placed 97,000 cu. yd. of concrete and had used 4.9 bags of cement per yard. The preliminary tests were made on the aggregates and cement used on this structure and we designed for 2480 lb. The result of our tests show of 257 specimens that have been broken at the 28-day age, an average strength—with no eliminations of specimens for anything—of 2410 lb. I might say that, figuring that ordinary loose volume requires 5.6 bags per cu. yd., we would save about 0.7 of a bag per yard of concrete in this structure, which amounts to about 38½ cents a yard at the price we are paying for cement. **Mr. Munsell.**

Characteristics of Concrete for Fire Resistance

Report of Committee E-4

That reinforced concrete has a high degree of resistance to building fires has been demonstrated by its performance in service. There are enough exceptions to its good record, however, to show that some precautions are necessary in the construction of reinforced concrete buildings intended to resist fires. The degree of fire resistance that must be attained for satisfactory service will depend, of course, on the severity, duration, and extent of the fire which may attack the structure. So far there are comparatively few data from which these fire characeristics may be determined. The best information available is contained in reports of fires in buildings[1] and some data of more specific nature derived from the tests at the National Bureau of Standards to determine the intensity and duration of fires in buildings.[2] These tests are still in progress but have already given data of value in comparing actual fires with testing procedure. These comparisons can now proceed with somewhat greater assurance of uniformity for within the last few months an American tentative standard has been established whereby the severity of fire effects on materials and construction may be determined by tests. And from test results experienced persons can judge the severity of building fires by comparison.

Contrasted with the meager quantitative data on the severity and duration of fires in buildings the results of a great many fire tests of concrete structures and structural members are available[3] from which the fire resistance of concrete has been determined. These tests have not been made with any view to determine the effect of the cements used but rather the fire resistance of particular details of the constructions or the constituent aggregates of the concretes. It is probable that the differences in the cements used in most fire tests have been insignificant as far as their behavior influenced the results.

The tests of W. A. Hull, formerly of the Bureau of Standards,[4] indicated that small amounts of hydrated lime added to concrete gave unfavorable results in fire tests, and Messrs. Stradling and Brady of the Department of Scientific and Industrial Research of England,[5] found that lime as a constituent of the cement influenced the residual strength of concrete after heating very materially. They drew a tentative conclusion that for satisfactory fire resistant concretes the cement should be selected or should have some admixture which would properly combine with the lime in the wet state to negative in part its ill effects when the resulting con-

crete is heated. In their tests a blast furnace slag portland cement without further admixture and an ordinary British portland cement with admixtures of finely ground steam boiler coal clinkers or with finely ground clay which had been burnt to a proper degree produced concretes which retained approximately half their strength after being heated to 1000 deg. C. (1832 deg. F.). The most suitable aggregates used were a red brick and a clay burnt to 850 deg. C. Each was used for both the fine and coarse aggregates. The best of the stone aggregates used, dolerite, produced a concrete of somewhat greater initial strength but of lower strength after heating. These concretes when wetted after burning regained in large part the strength lost. Other admixtures and aggregates will likely be found which behave similarly.

Diatomaceous earth has properties such that admixtures of small proportions should give favorable results in effecting greater fire resistance. On the other hand, lime, which has the same characteristics in effecting workability of concrete but in less degree, has been found to give unfavorable results because of the great changes taking place on heating. It is believed that only in exceptional cases will it be necessary to select a cement or use special admixtures to secure greater fire resistance because the choice of aggregates determines this characteristic to a great degree.

Several series of tests have been made with aggregates as the variable. These have resulted in giving more definite information with respect to them as affecting the fire resistance of concrete. Of the natural aggregates in common use the highly siliceous gravels are the least satisfactory in fire resisting properties. Concretes made from them have good strength before heating, but spall badly in fires and lose strength rapidly. The heat conductivity of such concretes is higher than for most of the concretes of the same mix, made from other natural aggregates.

Building members of gravel concrete such as columns and beams can be made to resist moderate fires fairly well by having greater thickness of protecting concrete with light metal mesh reinforcement embedded near the surface. Floor slabs will also require increased thickness of protection to the reinforcement but generally will not require metal mesh unless exposed in large areas. Portland cement or gypsum plasters, if bonded so as to stay in place, give satisfactory protection to gravel concrete members. Lime plasters and lime-cement plasters usually fall away promptly when exposed to fire. They can not be expected to afford protection unless applied on metal or wire laths which have been properly attached. Tests of gravel concrete with protective concrete of greater fire resistance have not proven as satisfactory as the reinforced protection. The calcareous gravels are much better in fire resisting properties than the siliceous gravels but are dependent, of course, on the combination and proportions of the calcareous and other mineral constituents.

Chert, flint and granites, like the siliceous gravels, produce concretes which spall when exposed to fire. Some of these may be even more unsatisfactory than the gravel. Concretes, made with them, protected in the

same manner as described for the gravel aggregates, should give satisfactory results. It is probable that more care will be necessary in providing protection for the chert and flint concretes. The spalling types of aggregates should be of small size for the best results. Larger aggregate particles are much more destructive to the concrete.

Most of the sandstones are unsatisfactory as aggregates with regard to fire resistance on account of shrinkage and loss of strength of the concretes in which they are used. The heat transmission through such concrete is not as rapid as through the siliceous gravel concrete but compared to most other concretes is faster. Extra thickness of protective concrete will usually be all that is necessary to provide for satisfactory resistance to moderate fires.

Trap rock and limestones are the best of the ordinary natural aggregate for fire resisting concretes. Limestone concretes resist fires longer and have a lower rate of heat transmission than the trap rock concretes. The latter, however, show less surface disintegration and therefore will usually require less surface repairs after being subjected to fire. The record of both in fires and tests has been satisfactory. The amorphous and finely crystalline rocks are generally more resistant to fire than the coarsely crystalline ones, and likewise the smaller aggregates are somewhat better than the coarse.

Concrete made with pumice for both fine and coarse aggregates has a slower rate of heat transmission and seems to be weakened less at a given temperature than either the limestone or trap rock concrete. It also has much less weight and lower initial strength for a particular mix than any of the foregoing concretes, but would be satisfactory for the fireproofing of steel structures, for floor slabs of moderate spans and for walls. The economy of weight for a multistory building is very much in its favor.

The two most commonly used artificial aggregates are cinders and blast furnace slag. Contrary to a quite general impression cinder concrete, of all the concretes, is relatively poor for insulating against fire. It transmits heat somewhat more readily than the siliceous gravel concretes and while it does not spall it usually suffers great loss of strength. Because of the high rate of heat transmission the thickness of protective coverings should be greater than for most of the other concretes. Cinder concrete in actual service, so far as the records disclose, has not been unsatisfactory. This is probably due to its use only in relatively minor members of structures and as fireproofing for structural steel. Clean hard burned clinkers as aggregates give much more favorable results than cinders. The heat transmission is less and the residual strength proportionately much greater. Coke breeze as an aggregate has been found in tests to make concrete of good heat insulating properties but is not satisfactory for reinforced concrete subjected to severe fires.

Blast furnace slag concrete has a fire resistance ranging between limestone and trap rock and should be satisfactory from this standpoint for all ordinary buildings. The rate of heat transmission through it is

higher than for limestone concrete and ordinarily it will suffer somewhat greater loss of strength, but in these respects it is slightly better than trap rock concrete in so far as the tests disclose. Like most all aggregates there are variations in composition and properties which may cause rather wide variations in results when slag concretes are exposed to fire but, in general, they may be classed with the limestone and trap rock concretes in fire resisting properties.

Burnt clay aggregates have been used in comparatively small amounts, but where fire resistance is of great importance they will be found more satisfactory than almost any of the other aggregates. Such concretes suffer less loss of strength than most of the others for a given rise of temperature and are relatively good insulators against heat transmission. It is important, however, that such aggregates be properly burned—neither too hard nor too soft—for satisfactory fire resisting properties of the concrete. The vesicular or light-weight burnt clay aggregates appear to be somewhat better in some respects than the solid materials.

They have many of the properties of pumice and concrete made from them should have nearly the same heat insulating and fire resistive properties. For a given weight of concrete they must have some advantage over the solid burnt clay aggregates. Burnt clay used for both the coarse and the fine aggregate is better than when used with sand as the fine aggregate. Concrete made with crushed fire brick aggregates has been satisfactory for frames for fire tests of walls where exposures have not exceeded six or eight hours. It does not have great strength but stays in place well.

None of the concretes made with Portland cement can be classed as highly refractory. Usually they will be found unsatisfactory for any service where exposed frequently or for long periods to temperatures of 1650 deg. F. (899 deg. C.) or above. Sometimes, however, concretes have been used in such locations with good results, but not uniformly so.

The reports of fires in concrete buildings indicate that frequently poor design and careless workmanship are responsible for more of the resulting damage than the unsuitability of the materials used. All the care that may be bestowed on the selection of the aggregates and cement to be used for a fire resisting structure can be negatived by either poor design or indifferent workmanship. Sometimes both are found as an insuperable combination against all the good intentions that may have existed relative to a worthy project.

The report of Committee E-4 for 1925 included certain admonitions relative to the design of reinforced concrete buildings with the view to minimizing damage in case of fire. These were concerned with the details of members only and did not discuss the more general problem of designing against fires in buildings.

All building materials and constructions have limits beyond which they can not survive in fires, therefore, it is important that buildings housing great quantities of combustibles be designed to limit fire spread.

This can be done by subdividing them with incombustible and fire resistive floors, walls and partitions with suitable fireproof doors and windows so as to confine fires to the room or unit in which they originate, thereby lessening both the intensity and duration of the exposure.

Briefly summarizing the foregoing we find that the characteristics of concrete for fire resistance are such that most concretes resist building fires well. The constituents of the cement affect the residual strength of concrete after fire exposure, but generally are of less importance than the kinds of aggregate used. Lime as an admixture gives unfavorable results in fires. Highly siliceous rocks and gravels which cause spalling are the most unsatisfactory and the limestones and trap rock the best of the common natural aggregates. Pumice appears to have a higher resistance but is not commonly used. Cinder concrete at high temperature transmits heat rapidly and loses strength. Blast furnace slag concrete is comparable in fire resistance to limestone and trap rock concretes. Concretes made of crushed well burnt bricks, fire bricks, or burnt clays lose strength in less proportion than most of the others. Vesicular burnt clay aggregates compare with pumice in the fire resisting properties. Care in mixing and placing concrete is as important in securing fire resistance as the selection of materials. The correctness of design of structures and details often accounts for the difference between satisfactory and unsatisfactory fire resistance. It is important that buildings housing combustibles be planned to limit fire spread, for all building materials and constructions have limits beyond which they can not survive.

N. D. MITCHELL, *Chairman.*

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[1] Some of the more informative reports of fires are contained in the following:

"Baltimore Fire" (1904). *Engineering News*, Vol. 51, pp. 145, 258, 261, 284; Vol. 51, pp. 276–331; Vol. 51, pp. 516–28, June, 1904. Bulletin XIII, Insurance Experiment Station. C. E. Norton. Report of National Fire Protection Association Committee. "The Baltimore Conflagration" (1904).

A Record of the Baltimore Conflagration, British Fire Prevention Committee, Journal 1.

"The San Francisco Earthquake and Fire" (1906). Bulletin No. 324 of United States Geological Survey. Report of S. A. Reed, Consulting Engineer, National Board Fire Underwriters. *Trans.* American Soc. Civil Engineers, Vol. 58, p. 206 (1907).

"Edison Factory Fire" (1914). *Engineering Record*, Vol. 70, p. 635, Dec. 12, 1914, p. 660, Dec. 19, 1914; Vol. 71, pp. 239–42, Feb. 20, 1915. Report of National Fire Protection Association. L. C. Wason, *Textile World*, Vol. 48, p. 537, Feb., 1915. *Engineering News*, Vol. 73, p. 362, Feb. 18, 1915. Report of Committee American Concrete Inst., Vol. 3, No. 8, Aug., 1915.

"Fire at Millenium Mills, London" (1917). Red Book No. 208, British Fire Prevention Committee.

"Fire at Quaker Oats Co., Peterboro, Ontario" (1916). *Engineering News*, Vol. 78, pp. 17–21, Apr. 5, 1917. Red Book No. 225, British Fire Prevention Committee. *Journal* Western Society of Engineers, Vol. 22, pp. 509–39, October, 1917.

"Fire in Concrete Warehouse at Galveston" (1920). *Engineering News*, Vol. 85, pp. 980–83, 1101–02, 1298. Vol. 86, p. 353, Dec. 2, 1920, Dec. 30, 1920, Feb. 24, 1921.

"Fire in Factory in Berlin, Germany" (1922). *Engineering News*, Vol. 88, pp. 795–96, May 11, 1922.

[2] "Fire Tests with Office Occupancies." *Quarterly*, National Fire Protection Association, Vol. 20, No. 3, p. 243, Jan., 1927. *Safety Engineering*, Vol. 53, No. 1, p. 29, Jan., 1927.

"Severity, Duration and Control of Exposure," by S. H. Ingberg. *Proc.* National Fire Protection Association, Vol. 31, p. 295, 1927.

"Severity of Building Fires," by S. H. Ingberg. *Proceedings* Fourteenth Annual Meeting Building Officials Conference, 1928, p. 87. *Safety Engineering*, August–September, 1928. *Quarterly* National Fire Protection Association, Vol. 22, No. 1, p. 43, July, 1928.

"Garage Tests." *Quarterly* National Fire Protection Association, Vol. 20, No. 1, July, 1926. Technical News Bulletin of the Bureau of Standards, July, 1926.

[3] Bureau of Standards Technologic Papers No. 184, Fire Tests of Building Columns, and No. 272, Fire Resistance of Concrete Columns.

British Fire Prevention Committee, Fire Tests of Concrete, Red Books Nos. 101, 212, 213, 216, 217, 218, 219, 224, 228, 237, 243, and 249 on plain concrete slabs; Nos. 221, 222, 223, 226, 227, 229, 231, 232, 234, 238, 239, 242, 244, 246, 248, and 250, on reinforced concrete slabs; Nos. 251 and 252 on conductivity of heat through slabs; No. 256, geologic notes and description of aggregates; No. 257, mechanical tests of concretes; and No. 258, a very brief synopsis of the tests.

Fire Tests of Floors, Woolson and Miller. *Proc.* Int. Assn. for Testing Materials, Paper XXVII, 2 (2d section), 1912.

Brandproben an Eisenbetonbauten, 1911–1915. Deutscher Ausschuss für Eisenbeton Heft 33, 66 pp. (1916). An elaborate series of fire tests on buildings of reinforced concrete two stories in height, using several types of aggregates.

Fire Resistance of Different Concretes, *Engineering Record*, Vol. 2, p. 97, July, 1905. Tests at Underwriters' Laboratories, Chicago, with gravel, limestone, granite and cinder concrete.

Investigation of the Thermal Conductivity of Concrete and Effect of Heat upon its Strength and Elastic Properties, Ira H. Woolson.

Proc. Amer. Soc. for Testing Materials, Vol. 6, p. 433 (1906), Vol. 7 (1907). *Eng. News*, June 28, 1906; Aug. 15, 1907.

Heat Insulating Properties of Some Materials used in Fire Resistive Construction. Technologic Paper No. 130, Bureau of Standards (1919). Includes concretes of 1:2:4 and 1:3:6 mixes and of cinders, clinker, gravel, limestone and trap rock aggregates.

[4] Heat Insulating Properties of Some Materials used in Fire Resistive Construction. Technologic Paper No. 130, Bureau of Standards (1919). Includes concretes of 1:2:4 and 1:3:6 mixes and of cinders, clinker, gravel, limestone and trap rock aggregates.

[5] "Fire Resistant Construction." Special Report No. 8, Department of Scientific and Industrial Research, London, England. Includes researches on the fire resisting properties of cements and aggregates.

A Study of Chapter 11

"Tentative Building Regulations for Reinforced Concrete"

By Phil J. Markmann*

Reasoning from Sec. 1108, the intended specification for the permissible load on reinforced-concrete columns may be stated thus:

1. The permissible axial load P of a reinforced-concrete column of the "spirally hooped" type is to be computed by equation 22, with the proviso that the load P so computed is to be reduced to $P' = P\left(1.50 - \frac{h}{100\,R}\right)$, equation 26 of Sec. 1108*a*, when the length is > 50 times "the least radius of gyration of the column core" ($50\,R$).

2. The permissible axial load P of a reinforced-concrete column "with lateral ties" is to be computed by equation 23, with the proviso that the load P so computed is to be reduced to $P' = P\left(1.33 - \frac{h}{120\,R}\right)$, equation 26*a* of Sec. 1108*b*, when the length is > 40 times "the least radius of gyration of the column section" ($40\,R$).

Note: The radius of gyration R in equations 26 and 26*a* computed as directed in Sec. 1108*c* is not the radius of gyration R_o of the *real* cross-section A, but of a *hypothetical* cross-section with pA increased to nAp. When A is either the core area of diameter "d" of a "spirally hooped" column or the over-all area of the "tied" column, the latter generally of square or rectangular figure with "a" the side of the square or the smaller side of the rectangle respectively, then

$$\text{Concrete area} + \text{steel area} = A(1 - p) + Ap = A$$

Transforming the steel area Ap into the larger area nAp gives the "hypothetical" area

$$A_1 = A\,(1 - p) + nAp = A[1 + (n - 1)p] \qquad \ldots\ldots\ldots\ldots\ldots\ldots\; a$$

The so-called radius of gyration of the cross-section of a column enters as a factor in computing its permissible axial load when we have to make allowance for bending stresses which develop in resisting buckling. The bending moment M resisted by the moment of the internal stresses

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of a homogeneous beam and, therefore, of a *plain-concrete* column of cross-sectional area A, whose so-called moment of inertia is I, is given by

$$M = s \times I \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots b$$

$$\text{or } M = s \times R_o^2 \times A \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots c$$

where s is the unit stress at the unit distance from the neutral axis, and R_o the "true radius of gyration" of the area A.

$$R_o = \sqrt{\frac{I}{A}} = \frac{d}{4} \text{ for a circular area}$$

$$= \frac{a}{\sqrt{12}} \text{ for a square or rectangular area}$$

This resisting moment $s \times I$ of the given cross-section A varies with the value of s, i.e., it is greater for a stronger material and smaller for a weaker material.[1] It is obviously true that a *reinforced-concrete* column of area A offers a greater resistance to bending (buckling) than a *plain-concrete* column of same area A. Its greater resisting moment is due to its greater s, in equations b and c. The adoption of the hypothetical radius of gyration R (of equations g and j later on) is based on conceiving the reinforced-concrete as a quasi-homogeneous material of a permissible stress $[1 + (n - 1)p]$ times the permissible stress s of the plain concrete. Therefore, with this conception, the resisting moment M_1 of the *reinforced-concrete* column of the same cross-section A having the same moment of inertia I (because I is independent of the material of which A is a cross-section) is

$$M_1 = s[1 + (n - 1)p] \times I \ldots\ldots\ldots\ldots\ldots\ldots d$$

$$\text{or } M_1 = s[1 + (n - 1)p] \times R_o^2 \times A \ldots\ldots\ldots\ldots e$$

In the two above equations $s[1 + (n - 1)p]$ is the unit stress at the unit distance from the neutral axis, the force of a hypothetical force couple, and I, or $R_o^2 \times A$, is its arm. We may detach the coefficient $[1 + (n - 1)p]$ from s and make it a coefficient of R_o^2, in equation e, without changing the right member of equation e, writing

$$M_1 = s \times R_o^2[1 + (n - 1)p] \times A \ldots\ldots\ldots\ldots f$$

In equation f, s is now a hypothetical unit stress at unit distance, while $R_o^2\,[1 + (n - 1)p]$ is the second power of a hypothetical radius of gyration. Designating the latter as R, we have

[1] Incidentally let it be pointed out that in equation b (and c also) the resisting moment may be construed as the moment of a hypothetical force couple, the force being s (the unit stress at unit distance from the neutral axis) and the arm of the couple being I (the so-called moment of inertia of the cross-section). This I spoken of as "a four-dimensional quantity," or as "biquadratic inches" in some text books, and marked "inches^4" in steel handbooks, thus reveals itself as "a simple length," viz., the arm of the said hypothetical force couple "s." The writer has called attention to this obvious definition of "the moment of inertia of the cross-section" many years ago. Yet tradition compels us to call this (hypothetical) length "the moment of inertia of the cross-section" when it is not anything like it. The "radius of gyration of the cross-section" is another such mis-applied term in Statics.

$$R = R_0 \sqrt{1 + (n - 1)p} \quad \ldots\ldots \quad g$$

Therefore, $M_1 = s \times R^2 \times A \quad \ldots\ldots \quad h$

The hypothetical radius of gyration R of equation g, and in equation h, is the R for equations 26 and 26a in Sec. 1108, because, when computed according to instructions in Sec. 1108c, it is the radius of gyration of the *hypothetical* area $A_1 = A[1 + (n - 1)p]$ of equation a, viz., when A is the *real* (circular) core area of diameter d of a spirally hooped column, then A_1 is the *hypothetical* (and also circular) area after transforming the steel area Ap into an area $n \times Ap$. The diameter d_1 of the hypothetical area

$$A_1 = A[1 + (n - 1)p] \text{ is } d_1 = 2\sqrt{\frac{A_1}{\pi}} = 2\sqrt{\frac{A}{\pi}} \times \sqrt{1 + (n - 1)p}$$

or since $2\sqrt{\dfrac{A}{\pi}} = d$

$$d_1 = d\sqrt{1 + (n - 1)p}$$

and the radius of gyration of this *hypothetical* circular area is

$$R = \frac{d_1}{4} = \frac{d}{4}\sqrt{1 + (n - 1)p} \quad \ldots\ldots \quad i$$

For square or rectangular areas, a being the side of the square, or the smaller side of the rectangle, respectively, we have

$$R = \frac{a}{\sqrt{12}} \sqrt{1 + (n - 1)p} \quad \ldots\ldots \quad j$$

It would be proper and helpful to incorporate equations i and j in Sec. 1108c, they give the respective values, for circular and rectangular cross-sections, of the hypothetical radius of gyration R of equations 26 and 26a so definitely that there can be no fumbling.

Now, as to Sec. 1101a, we would expect it to say: The permissible axial load of *reinforced-concrete* columns of the spirally hooped type, when not longer than 50R is to be computed by equation 22 of Sec. 1103, and the permissible axial load of *reinforced-concrete* columns with separately tied longitudinals, when not longer than 40R, is to be computed by equation 23 of Sec. 1104. [Note: The R in either case is to be computed by equations i and j, respectively, to be added to Sec. 1108.] But Sec. 1101 says: "Reinforced-concrete columns" (making no distinction between types) "shall not be longer than eleven times the least lateral dimension, unless designed as long columns under the provisions of Sec. 1108." Thus we have two scales for measuring the length of the column, and it is evidently believed that a spiral column 50R long and a tied column 40R long is of the same length as 11 times their least lateral dimensions, at least approximately so; but there is quite a difference, as will be shown later on.

The inference from equation 26 and 26*a* is that (for spirally hooped columns) bending stresses may be ignored when the column length is $\leqq 50R$, and (for tied columns) bending stresses may be ignored when the column length is $\leqq 40R$.

Example 1:

A spirally hooped column of core diameter $d = 22$ in., $p = .03$, $n = 15$. The hypothetical radius of gyration, by equation *i* is

$$R = \frac{d}{4}\sqrt{1 + (n - 1)p} = \frac{22}{4}\sqrt{1.42} = 5\tfrac{1}{2} \times 1.19 = 6.54 \text{ in.}$$

Therefore, $50R = 50 \times 6.54$ in. $= 327$ in. $= 27$ ft. 3 in. is, by Sec. 1108*a* the limit of length which P by equation 22 must be reduced. But, taking "the least lateral dimension" in Sec. 1101 to be the full diameter (25 in.) of this column (including a 1½ in. thick protective shell all around), we would have

$$11 \times 25 = 275 \text{ in.} = 22 \text{ ft. } 11 \text{ in.}$$

as the limit of length for the use of equation 22, so that the column length from 22 ft. 11 in. to 27 ft. 3 in. is not covered by either equation 22 or equation 26. This gap would not exist if the limit of the column length in Sec. 1101*a* were specified as $50R$ (for this type of column), instead of 11 least lateral dimensions.

Example 2:

A "tied" column of side "a" when square, or smaller side "a" when rectangular, $p = .02$ (the stipulated max. p for such columns) and $n = 15$. The hypothetical radius of gyration, by equation *j* is

$$R = \frac{a}{\sqrt{12}}\sqrt{1 + (n - 1)p} = \frac{a}{3.464}\sqrt{1.28} = \frac{a}{3.464} \times 1.13 = .326a$$

Therefore, $40R = 40 \times .326a = 13.04a$ is, by Sec. 1108*b*, the limit of length beyond which P by equation 23 must be reduced, but by Sec. 1101*a* we have $11 \times a$ as the limit of length for the use of equation 23, so that the column length from $11 \times a$ to $13.04 \times a$ is not covered by either equations 23 or 26*a*. This gap would not exist if the limit of the column length in Sec. 1101*d* were specified as $40R$ (for this type of column), instead of 11 least lateral dimensions.

It is not clear why a difference should be made in the length limitation —$50R$ for spirally hooped columns and $40R$ for tied columns. The 1924 Specifications made $40R$ the "no buckling stress" limit for all reinforced-concrete columns, and the writer thinks equation 47 in the said Specifications, being the same as equation 26*a* in these present "Tentative Building Regulations" should be made to apply to spirally hooped columns as

well as tied columns. It would be on the side of safety, and in the case of the first example above would give

$$40R = 40 \times 6.54 \text{ in.} = 262 \text{ in.} = 21 \text{ ft. } 10 \text{ in.}$$

as the limit of length for this particular column, and this length would come much nearer to 11×25 in. = 275 in. = 22 ft. 11 in., the limit according to Sec. 1101*c*. The double standard for measuring the column length is unnecessary and confusing. Make R the scale for measuring the length of the column, and have for spiral or tied, columns $\lesseqgtr 40R$ in length, equations 22 or 23, respectively and for spiral and tied columns $>40R$ in length, equation 26*a*.

The hypothetical R, as is apparent from equations i and j, varies for the same size column with p and n:

R of the spirally hooped 22 in. core column being as shown in the following tabulation with R increasing as p increases and decreasing as n decreases:

	p					
	.01	.02	.03	.04	.05	.06
$n = 15$............	5.90	6.20	6.54	6.87	7.15	7.41
$n = 12$............	5.76	6.05	6.33	6.60	6.80	7.10
$n = 10$............	5.73	6.00	6.20	6.44	6.60	6.80

Comment on Sec. 1103:

Equation 22*a* is an empirical formula for the f_c of equation 22 (spiral columns), and, furthermore, Sec. 1103*c* dictates a fixed ratio between p', the ratio of the spiral reinforcement and p, the longitudinal steel ratio, viz.:

$$p' \gtreqless \frac{p}{4}, \text{ or } p \lesseqgtr 4p'$$

Any qualified designer will feel at liberty to use, say with $p' = .005$, any longitudinal steel ratio p from say .01 to .06 and, conversely, he will feel at liberty to bind a reinforced-concrete column of $p = .01$ to .06 with any hooping of say $p' = .005$ to .015.

The rational equation for the permissible axial load of a spirally hooped column is

$$P = A \times kf_c \times [1 + (n - 1)p] \qquad \ldots\ldots\ldots\ldots \quad l$$

where f_c say $= \frac{f_c'}{4}$ is the permissible stress of the *non-hooped* concrete, and

the coefficient k is a function of p' and n, so that kf_c is the enhanced permissible compressive stress of the *hooped* concrete. By the Chicago code, for instance,

$$k = 1 + 2.5\,np' \ldots\ldots\ldots\ldots m$$

The permissible stress, kf_c increases with p', for a given concrete. Possibly authentic tests on record may establish k as a function of p' and n different from $1 + 2.5\,np'$. But whatever this function is, equation l, is the rational equation for P. With the permissible stress f_c of a given non-hooped concrete increased by hooping to the permissible stress kf_c the unit stress in the longitudinal steel, whatever its ratio p, is automatically nkf_c. Any p may be associated with any p', and vice versa, within reasonable limits, say $p' = .005$ to $.015$ and say $p = .01$ to $.06$.

A given load P, by equation l, is the product of three factors, viz., A, kf_c and $[1 + (n - 1)p]$, all three variable at will, independent of one another. A certain size column may answer for a limited variation of load by varying either kf_c or $[1 + (n - 1)p]$, or both. A certain kf_c will answer for a variation of load by varying either A, or $[1 + (n - 1)p]$, or both. A certain $[1 + (n - 1)p]$ will answer for a variation of load by varying either A, or kf_c, or both. This is the way the practical designer has to and always will design a group of columns which, though the loads vary some, may be desired to be of the same finished size, and can be made so by varying the hooping p' and the longitudinal steel ratio p, so as to suit the respective loads of the several columns of the set.

When p in equation 22a is taken as ° (which implies $p' =$ ° also) the said equation must be expected to give the permissible stress of the *non-hooped* concrete, or the permissible *basic* stress designated say by f_o:

For a 2000 lb. concrete—$f_o = 300 + 200 = 500$,
or 25/100 of the ultimate strength f_c'

For a 2500 lb. concrete—$f_o = 300 + 250 = 550$,
or 22/100 of the ultimate strength f_c'

For a 3000 lb. concrete—$f_o = 300 + 300 = 600$,
or 20/100 of the ultimate strength f_c'

In other words, with a stronger concrete the permissible basic stress f_o would be a smaller fraction of the ultimate strength. Why should we make this inconsistent stipulation?

The f_c of equation 22 corresponds to the kf_c of equation l above. Equation 22a, substituting $4p'$ for p may be written:

$$f_c = 300 + (.10 + 16p')f_c' \ldots\ldots\ldots\ldots n$$

The Chicago code values kf_c and the values f_c (from equation 22*a*) are tabulated for comparison:

Comparison of $kf_c = \frac{f_c'}{4}[1 + 2.5\, np']$ of Chicago Code with

$$\left.\begin{array}{r} f_c = 300 + [.10 + 4p]f_c' \\ p = 4p' \end{array}\right\} \text{Equation 22}a\text{—A.C.I.}$$

	$f_c' = 2000$, $n = 15$	P′ .0025	.005	.0075	.01	.0125	.015
Chicago	kf_c	....	594	640	688	734	780
	Per cent of ultimate strength f_c'	....	30%	32%	34.4%	36.7%	39%
A.C.I.	f_c	580	660	740	820	900	980
	Per cent of ultimate strength f_c'	29%	33%	37%	41%	45%	49%
	$f_c' = 2500$, $n = 12$						
Chicago	kf_c	....	719	766	813	859	906
	Per cent of ultimate strength f_c'	....	28.8%	30.7%	32.6%	34.4%	36.2%
A.C.I.	f_c	650	750	850	950	1050	1150
	Per cent of ultimate strength f_c'	26%	30%	34%	38%	42%	46%
	$f_c' = 3000$, $n = 10$						
Chicago	kf_c	....	844	891	938	984	1031
	Per cent of ultimate strength f_c'	....	28.1%	29.7%	31.3%	32.8%	34.4%
A.C.I.	f_c	720	840	960	1080	1200	1320
	Per cent of ultimate strength f_c'	24%	28%	32%	36%	40%	44%

Can we conscientiously advocate a f_c for equation 22 as high as 820 lb. in a column in which $p' = .01$, when this f_c is as much as 41 per cent of the ultimate strength $f_c' = 2000$ lb. while the permissible concrete stress for this $p' = .01$ by the Chicago code is only 688 lb. or 34.4 per cent of the said ultimate strength?

Can we conscientiously advocate a f_c for equation 22 as high as 980 lb. in a column in which $p^1 = .015$, when this f_c is as much as 49 per cent of the ultimate strength $f_c' = 2000$ lb. while the permissible concrete stress for this $p' = .015$ by the Chicago code is only 780 lb. or 39 per cent of the said ultimate strength?

As not many probably are aware of the origin of the strange formula 22*a*, let it be said: it is an outgrowth of the *supposed* relation of the steel and concrete stress in the reinforced-concrete column, *when under load*, given as

$$f_s = nf_c + Em$$

(and so recorded) on p. 166 of the 1921 A. C. I. *Proceedings*. Note that this equation says—the steel stress f_s is always—*not* nf_c, the modular ratio, *but* $nf_c + 15{,}000$ (when E is taken as 30,000,000 and $m = .0005$). The author of that paper arrived at his truly amazing result after stating and formulating three successive misconceptions relating to the action and effect of external load on the concrete and the steel in the column. Proof of the writer's assertion will be found in his paper, pp. 126 to 166, 1927 A.C.I. *Proceedings*, and pp. 424 to 431, 1928 A.C.I. *Proceedings*. Special attention has been given to the 1921 paper on pp. 140 to 142 in the 1927 A.C.I. *Proceedings*.

This formula 22*a* appeared as formula 43 in the 1924 Joint Committee Specifications and is transferred to these present "Regulations," *just as if nothing had happened in the meantime*. Formula 22*a*, being based on a conclusion from a number of false premises, should be scrapped forthwith and replaced by

$$f_e = \frac{f_c'}{4}(1 + 2.5\,np') \qquad \ldots\ldots\ldots\ldots\ldots\ldots \quad o$$

in which the coefficient 2.5 of np' may be subject to revision.

RÉSUMÉ OF THE WRITER'S OBJECTIONS AND RECOMMENDATIONS

1. State the maximum permissible length to which equations 22 and 23 are applicable in terms of the hypothetical radius of gyration R, equations i and j, making the maximum lengths of Sec. 1101 coincident with the minimum lengths of the "long columns" of Sec. 1108.
2. Equation 26*a*—for long columns—to apply to both "spiral" and "tied" type. Eliminate equation 26.
3. Give the definite formulae i and j for the hypothetical radius of gyration R in equation 26*a* (equation 26 assumed eliminated).
4. Eliminate the arbitrarily stipulated ratio $p' \gtreqless \frac{p}{4}$.
5. Replace the inept formula 22*a* by something of the form of equation o.

Mr. Sutherland. HALE SUTHERLAND (*By Letter*)—The worth of this study with its objections and recommendations is entirely dependent upon the validity of the definition of radius of gyration there given and upon the correctness of the arguments advanced by the same author in the 1927 *Proceedings* in a paper with the title "The Reinforced Concrete Column."

The equations i and j defining the author's conception of radius of gyration in no way involve the location of the column reinforcement. In effect the assertion is that a column with its reinforcing rods bunched along the longitudinal axis has the same lateral stiffness as though these bars were at the edge of the core in the usual manner. This is manifestly not in accord with the facts.

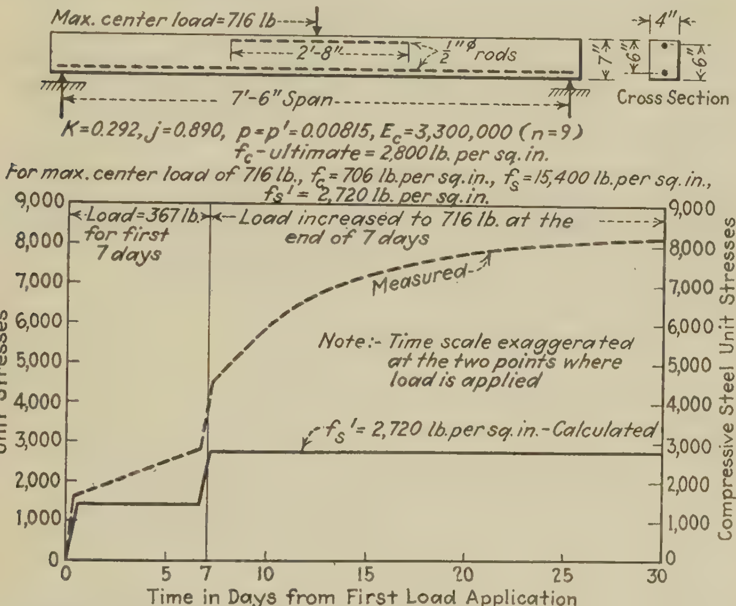
The hub of the argument of the 1927 paper is thus stated (Vol. 23, A.C.I. *Proceedings*, page 126–7): "In the reinforced concrete column the steel being bonded to the concrete by its compressive strength *tends* to resist the contraction of the concrete, but as the steel cannot be put in compression without engaging the concrete to cooperate with it, in conformance with Eq. 1 ($P/A = f_c[1 - (n - 1)p]$) the resistance to the contraction of the concrete is the resistance offered *by the steel in combination with the concrete.*" The obvious impossibility of this analysis has already been pointed out by others. It is sufficient here to remark that the well-known formula relating external load on a column to internal fibre stress, a relation based upon the assumption of the lack of initial stress in the materials, is here being used to evaluate initial stress in the absence of external load. In consequence the mathematical relations evolved have no physical meaning and no basis at all appears for the criticism (pages 140–142) made of Mr. F. R. McMillan's paper, "A Study of Column Test Data," 1921 *Proceedings*. The 1927 criticisms are repeated in the present paper but no new evidence is offered in their support.

Mr. Maney. G. A. MANEY (*By Letter*)—Mr. Markmann asks for a complete change of column specification in the "Tentative Regulations," ridiculing the fundamental concept of its basis. Such a procedure would surely mean turning back the wheels of progress in quest for a uniformly acceptable column code.

The "Tentative Regulations" concedes the established fact that shrinkage and plastic flow of concrete under load greatly affects and increases the unit stresses in all types of compressive steel.

Evidence is here offered from an experimental investigation of compressive beam steel under continued load which was finished recently in the structural laboratory at Northwestern University. The curves here

submitted are based on the average of two beams of this series which in general gave very close check results with duplicate specimens. The diagram compares a curve of calculated unit stress in the compressive steel based on the common theory of the "Tentative Regulations" with actual measured unit stress as indicated by the heavy dotted line. The measured increments of stress which occur immediately due to load application check well with calculated stresses but after a month of loading causing ordinary design stresses these values are almost trebled due to shrinkage and plastic flow.



Comparison of Stress in Compressive Steel Calculated on the Theory of the "Tentative Regulations" with Actual Measured Stress.

To be consistent the "Tentative Regulations" should make definite provision for this greatly increased efficiency of compressive beam steel since it already recognizes the fact of high unit stresses in column steel.

For those who feel with Mr. Markmann that formula (22a) should be "scrapped forthwith," a brief comparison with the Chicago Code is here submitted on the basis of a 2000-lb. concrete.

By the Chicago Code

$$\frac{P}{A_c} = 500\,(1 + 14p)(1 + 37.5p')$$

By "Tentative Regulations"

$$\frac{P}{A_c} = 500\,(1 + 14p)(1 + 16p')$$

It will be noted that only the last term differs and if we make $p' = 0.427p$ for both codes they become the same. The latter code provides that p' shall be equal to or greater than $0.25p$ so that any ratio larger than 0.25 would represent merely a conservative use of spiral. For higher percentages of vertical steel this would only mean a raising of the arbitrary upper limit of 1.5 per cent spiral in the Chicago Code. For the engineer who is aggravated by the column code of "Tentative Regulations" the Chicago Code is urged as a remedy to be taken as prescribed. Except for limiting steel percentages there is no case where the two codes cannot be shown to be an identity.

The last factors of the formulae above given express the difference in the point of view of the "Tentative Regulations" from that of most other codes. High stresses in vertical steel under design load conditions are emphasized rather than the increases in ultimate load offered by spiral steel.

The question here raised is whether our lower limit of spiral percentage and our ratio of minimum spiral to vertical percentages should not be raised considerably in the "Tentative Regulations." The demands of the future for gigantic structures in reinforced concrete must lead us to give spiral its due as the all-important dependability factor in the reinforced concrete column.

Discussion—Design and Cost Data for the 1928 Joint Standard Building Code[1]

H. M. Hadley and W. F. Way.[2]—Mr. Lord's very excellent and comprehensive paper presents to the Institute the results of a great amount of careful, painstaking, and tedious labor and entitles him to the thanks and appreciation of the entire membership. Whatever can be done to ease and simplify the work of design is of great help to the development of reinforced concrete. The apparently involved and forbidding calculations which its use entails have unquestionably led to the rejection of concrete and the use of other materials in numerous cases, despite the adaptability and even economy which reinforced concrete offers. Therefore, Mr. Lord's paper is particularly valuable, and his comments on simplicity and workability are worthy of serious consideration. **Mr. Hadley and Mr. Way.**

Mr. Lord makes reference to the elimination of waste which has resulted from the work of the Division of Simplified Practice of the Department of Commerce and has himself followed their recommendations with respect to reinforcing bar sizes. The writers feel, however, that attention should also and again be called to the elimination of waste that would result from designers' reducing to a minimum the number of beam sizes and slab spans and sizes in any particular job. How much tortured and torturing work has been done under the mistaken notion that it was economical to save material, regardless of the variety and complication of form work that resulted from the saving! The choice by the designer of beam breadths suitable for the use of commercial sized lumber should also be mentioned. Twelve in. wide beams are undesirable when 11½-in. widths can be made to serve and simplify form work. All of this was very happily summarized in the somewhat broken English of a Belgian engineer who said: "Always make your designs as plain and simple as may be, for they are for men whose business is not to study, but to put."

In the design of beams—rectangular, T-beams, Γ-beams, with or without compressive reinforcement—there is another simple method, which is submitted in the following pages. This method the writers developed in 1919, and at first, believed they had originated. However, in Hool and Johnson's Concrete Engineers' Handbook, p. 304, published in 1918, the same treatment of the problem of compressive reinforcement is credited to Robert S. Beard. The Reinforced Concrete

[1] A. C. I. *Proceedings*, Vol. 24, 1928, p. 537.

[2] Respectively, district engineer, Portland Cement Association, and engineer with Henry & McFee Contracting Co., Seattle, Wash.

DESIGN TABLES of Thomas and Nichols, McGraw-Hill, 1917, pp. 9 and 10, also handles compressive reinforcement in this same manner. So far as the writers know, this method of design of the ordinary T-beam has not hitherto been published.

In its workability, and in the unity of conception and thought with which *all* beams are treated, the method has much to commend it and it possesses the sovereign virtue of simplicity. Moreover it is a method of wide applicability, and the data which is required for the case of a concrete of 2000 lb. strength is equally applicable to the case of an 1800-lb. concrete, a 1900-lb. concrete, or one whose ultimate strength might be 2275 lb. per sq. in.

In other words, with any definite working stress in the reinforcement, all the concretes that are assumed to have a common value of n may be treated with a single set of functions. It is true that the formula

$$n = \frac{30{,}000}{f_c'}$$

which appears in the Proposed Standard Building Regulations would restrict and limit the application of the accompanying tables to only those same concretes to which Mr. Lord's tables apply, since under this new formula each concrete has its own individual and characteristic value of n. Under it an 1800-lb. concrete has $n = 16.67$; a 1900-lb. concrete has $n = 15.79$; a 2275-lb. concrete has $n = 13.19$. In contrast with this, all of these concretes under the Joint Committee Report of 1924 are assumed to possess moduli of elasticity whose common identical value is 15. Considering the more or less arbitrary nature of the assumptions involved in both cases (see "The Modulus of Elasticity of Concrete," by Stanton Walker, Bulletin No. 5, Structural Materials Research Laboratory, Lewis Institute) and the mathematical complexities consequent upon the adoption of the proposed formula for n, it seems rather unlikely to the writers that it will find wide acceptance, and much more probable that the Joint Committee values of n will continue to be generally used. It is under the latter assumption that the statement is made repecting the applicability of the accompanying tables to more than a few concretes.

Preliminary to the design of any beam of any material, some definite loading must be determined, and definite external shears and moments must be calculated. With structural steel and wood the simplest procedure for finding the beam required for any certain case is to divide this calculated moment by the working stress to be employed, thereby determining the value of the section modulus. This is in accordance with the basic, fundamental formula resulting from the assumption of straight line variation of stress; namely,

$$\text{Section modulus (S)} = \frac{\text{moment (M)}}{\text{maximum fiber stress } (f)}$$

Having determined the actual numerical value of the section modulus, then by reference to tabulated values of section moduli in steel or lumber

hand books, an actual definite beam, or beams, with section modulus equal to the calculated section modulus, is found; it is checked for shear, and the calculation is ended.

Now with reinforced concrete, with all the possibilities which are at the command of the designer for varying the concrete mixtures and thereby changing their strength and elastic properties, for selecting different grades of reinforcing steel, and for freely changing the dimensions of structural members to suit the necessities of any particular problem, there is presented a material whose flexibility and adaptability to different situations is unequalled. On the other hand it is this same versatility of reinforced concrete which makes the preparation of comprehensive design tables exceeding difficult, and it is unlikely that the same procedure used with steel and wood can ever be employed with concrete.

It is, however, easily possible to calculate, for a wide range of values of concrete stresses and with sufficient variations in the steel stresses and character of concrete to cover all practical field conditions, tables of values of K and p which result from different combinations of concrete stress (f_c), steel stress (f_s), and concretes of different elastic properties. The value K may be defined as the term by which the product of the breadth and the square of the depth—effective—of a rectangular concrete beam, must be multiplied to determine what moment it can resist with definite maximum stresses in steel and concrete, the concrete quality and character being defined and measured by the term n. The value p is the corresponding "percentage": the product of breadth and effective depth multiplied by it gives the area of reinforcement required.

With such tables, then, the problem of design consists of finding one or more beams whose moment capacities equal the external moment. If the beam is of rectangular section, the operation is direct since the division of the external moment, M, by the coefficient, K, corresponding to the design fiber stresses in steel and concrete and to the proper value of n results in a quotient which is equal to the product of the beam breadth multiplied by the square of the effective depth.

If the beam is of T- or Γ-section, its moment capacity is found by treating it as *the difference of two rectangular beams*, the first and larger of which has a breadth equal to the sum of the stem breadth and that of the flange projections, and a depth equal to the total effective depth, while the second and smaller has a breadth equal to that of the flange projections only and a depth equal to the total effective depth minus the flange thickness. The first and larger beam has its moment capacity determined by the maximum working stresses in both steel and concrete; the second and smaller, by the same maximum stress in the steel but by a concrete stress equal to that existing on the under side of the T-beam flange, as determined by the assumption of straight-line variation of stress. The reinforcement of the T- or Γ-beam is, similarly, the difference of the reinforcement of the two rectangular beams.

If the dimensions of rectangular, T-, or Γ-beams cannot well be

increased beyond some certain maximum limits, then for extreme conditions compressive steel may be required. How much moment must be carried by compressive steel is the difference between the external moment and the moment capacity of the beam as determined by the tables. With compressive steel introduced into the beam at some certain plane, the stress in it is equal to n times that in the surrounding concrete (assume $(n - 1)$ since compressive steel displaces an equal area of concrete). Its lever arm is the distance to the tension steel. The moment value of one square inch of compressive steel is equal to the product of unit stress by the lever arm. The required amount of compressive steel is found by dividing the total moment to be carried thus by the moment value of one square inch of compressive reinforcement. To balance the compressive steel, tensile reinforcement additional to the normal tensile reinforcement, is required. This additional tensile reinforcement is stressed at the normal maximum steel stress, f_s. Its amount, then, is a certain fraction of the compressive reinforcement, and is determined by the ratio of the stress in the compressive reinforcement to the stress in the tensile reinforcement.

The accompanying tables cover values of f_c, varying by 10-lb. intervals, from 100 to 1500 lb. per sq. in. with a steel stress of 20,000 lb. per sq. in. and values of n of 8, 10, 12, and 15. Similar tables may be calculated for steel stresses of 16,000 and 18,000 lb. per sq. in. for use in those places where city ordinances "take precedence."

The great range of values which such tables offer is, of course, apparent. The solution of all beams becomes mathematically exact. The stem of a T-beam, beneath the flanges, does not have to be neglected in the computations.

The method of solution and use of tables can best be illustrated by a few examples.

EXAMPLE 1.

Consider the beam in Mr. Lord's Problem 2 on pages 54 to 61 of Vol. 24, *Proceedings*.

Design at Center.—This beam is subjected to a midspan moment of 845,000 in. lb.

$f_c = 3000$ lb.
$n = 10$
d effective $= 13.5$ in.
b of stem $= 12$ in.
t of slab $= 4$ in.

By paragraph 706 of Proposed Standard Building Regulations "the overhanging width on either side of the web shall not exceed 8 times the thickness of the slab nor one-half the clear distance to the next beam." With 12-in. beams, 6 ft. o.c., the clear distance between beams equals 60 in., and therefore the overhanging flange width equals 30 in. Our beam then has a total b of 12 in. plus (2) (30 in.) = 72 in. and d effective = 13.5 in.

In such cases as this where it is not directly apparent whether the beam is to be figured as a rectangular or T-section it is first necessary to decide this point. If the neutral axis falls at the bottom of the slab, or in the slab, the beam is classed as rectangular, whereas if the neutral axis falls below the bottom of the slab it is a T-beam. If the neutral axis lies exactly at the bottom of the slab, the concrete stress (f_{cc}) at the top of the slab may be found by the following formula:

$$f_{cc} = \frac{(\ t\ )\ (f_s)}{(d - t)\ (n)}$$

$$= \frac{(\quad 4 \quad)\ (20{,}000)}{(13.5 - 4)\ (\ 10\ )} = 840 \text{ lb. per sq. in.}$$

Therefore, if the value of f_c as determined by dividing the external moment M by the total breadth and by the square of the effective depth, and from the tables, is equal to or less than the above critical value, the beam is to be figured as a rectangular section; if the value is greater, the beam is to be figured as a T-section.

Proceeding now with our problem and dividing M by bd^2

$$K = \frac{845{,}000}{(72)\ (182.25)} = 64.4$$

Referring to tables this value of K corresponds to a fiber stress of 605 lb. per sq. in. Since this value is less than the critical value of 840 lb. per sq. in., found above, the neutral axis lies above the bottom of the slab and the beam should be figured as a rectangular section.

From tables, the value of p corresponding to the f_c of 605 lb. per sq. in., is 0.0035.

Therefore $A_s = (0.0035)(72)(13.5) = 3.40$ sq. in.

In comparing this treatment with Mr. Lord's method it is not to the slight reduction in reinforcement that we would call attention but rather it is to the difference in the basic conceptions of stress distribution involved in the two methods. This is of particular importance to the young engineer. With the one method 24.2 in. of stem and flange are assumed stressed at 1200 lb. per sq. in. maximum, while the remainder of the total 72-in. breadth carried no stress; with the other method the full breadth is assumed to be subjected to an uniform maximum stress of 605 lb. per sq. in. throughout. It is probable that the assumption of the second method more closely and truly approximates the actual stress distribution in an average beam in a concrete floor system than does that of the first method (see Figs. 1 and 2).

Design at Support.—The moment at the support in Mr. Lord's problem is 1,124,000 in. lb. and at the support $f_c = 0.45f^1_c = 1350$ lb. per sq. in.

The moment capacity of the beam, without compressive reinforcement, is Kbd^2.

By tables

$$M \text{ (capacity)} = (235.6)\ (12)\ (182.25) = 517{,}000 \text{ in. lb.}$$

The moment capacity is so much less than the external moment that changes in beam dimensions are necessary.

Accepting Mr. Lord's new breadth of 20 in., the moment capacity with this increased breadth

$$M \text{ (capacity)} = (235.6)(20)(182.25) = 858{,}000 \text{ in. lb.}$$

for which

$$A_s = (0.0136)(20)(13.5) = 3.67 \text{ sq. in.}$$

1,124,000 in. lb. − 858,000 in. lb. = 266,000 in. lb., to be carried by compression steel, centered 2½ in. below the top of the slab.

The stress in the concrete at this plane f_{cs} is determined by the formula

$$f_{cs} = f_c - \frac{(t)}{(d)}\left(\frac{f_s}{n} + f_c\right)$$

$$\text{Evaluating, } f_{cs} = 1350 - \frac{(2.5)}{(13.5)}\left(\frac{20{,}000}{10} + 1350\right) = 730 \text{ lb. per sq. in.}$$

And the effective value of one square inch of compressive steel at this point equals $(n-1)(f_{cs})$. We use $(n-1)$ instead of n, because the compressive steel replaces its area of compressive concrete.

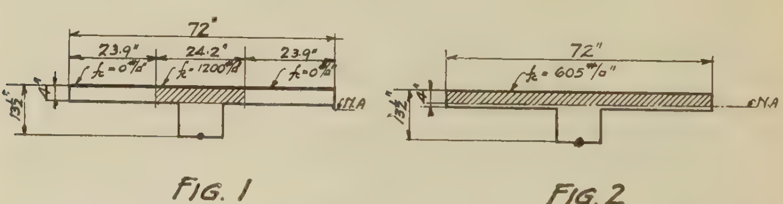


FIG. 1 AND FIG. 2.—STRESS DISTRIBUTION BY MR. LORD'S METHOD *vs.* UNIFORMLY OVER ENTIRE WIDTH OF FLANGE.

Here $(n-1)(f_{cs}) = (10-1)(730) = 6570$ lb. per sq. in.

This unit of compressive steel has a lever arm about the tensile steel of 13.5 in. − 2.5 in. = 11 in. and its moment value equals (6570 lb.) (11 in.) = 72.270 in. lb. Therefore, the compressive steel required in our problem equals

$$A_{cs} = \frac{266{,}000}{72{,}270} = 3.68 \text{ sq. in.}$$

The additional tensile steel required equals

$$A_{ts} = \frac{(n-1)(f_{cs})}{f_s}(A_{cs})$$

$$A_{ts} = \frac{(6570)}{(20{,}000)}(3.68 \text{ sq. in.}) = 1.21 \text{ sq. in.}$$

And the total tensile steel equals

$$3.68 + 1.21 = 4.89 \text{ sq. in.}$$

EXAMPLE 2.

A T-beam has a 5-in. slab framing on one side and a 4-in. slab on the other. As frequently occurs in building construction, due to finish floor conditions, the 4-in. slab is depressed 1½ in. below the top of the stem and the other slab. The effective depth is 28 in. (see Fig. 3). 2200-lb. concrete. $n = 15$. $f_s = 20{,}000$ lb. per sq. in. Design to resist an external moment of (1) 5,140,000 in. lb.; and (2) 8,350,000 in. lb.

By code $f_c = (0.40)\ (2200) = 880$ lb. per sq. in., which is the stress at the top of the stem and the 5-in. slab.

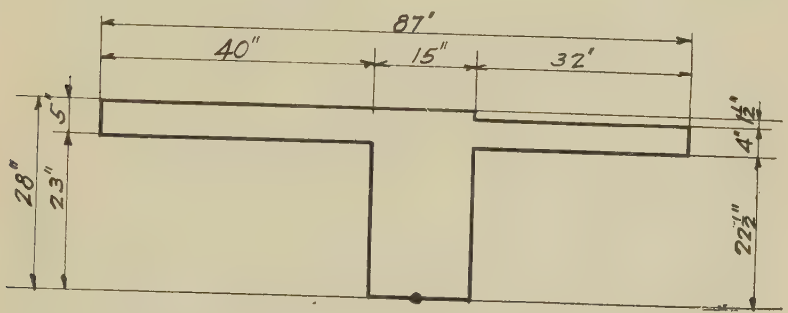


FIG. 3.—BEAM OF EXAMPLE 2.

Then concrete stress at top of 4-in. slab equals

$$880 - \frac{(1.5)}{(28\ )}\ (1333 + 880) = 760 \text{ lb. per sq. in.}$$

Concrete stress at bottom of 4-in. slab equals

$$880 - \frac{(5.5)}{(\ 28)}\ (1333 + 880) = 445 \text{ lb. per sq. in.}$$

Concrete stress at bottom of 5-in. slab equals

$$880 - \frac{(\ 5\ )}{(28)}\ (1333 + 880) = 485 \text{ lb. per sq. in.}$$

The moment capacity of this beam—without compressive reinforcement—equals, by tables,

+(151.8) (55) (784) =	+6,550,000	
−(58.9) (40) (529)		−1,245,000
+(120.9) (32) (702.25)	+2,720,000	
−(51.0) (32) (506.25)		− 825,000
	+9,270,000	−2,070,000
	−2,070,000	
	+7,200,000 in. lb.	

and tensile steel area equals

$+(0.00875)\ (55)\ (28)$	$= +13.48$ sq. in.	
$-(0.00323)\ (40)\ (23)$		-2.97 sq. in.
$+(0.00688)\ (32)\ (26.5)$	$+5.83$	
$-(0.00278)\ (32)\ (22.5)$		-2.00
	$+19.31$ sq. in.	
	$-\ 4.97$ sq. in.	
	14.34 sq. in.	

Therefore, the moment capacity is

$$7{,}200{,}000 \text{ in. lb. and } A_s = 14.34 \text{ sq. in.}$$

As a check on the above calculations

$$jd = \frac{M}{(f_s)\ (A_s)} = \frac{7{,}200{,}000}{(20{,}000)\ (14.34)} = 25.1 \text{ in.}$$

Then $j = \dfrac{25.1}{28} = 0.898$

This value of j, being approximately 0.90, shows that the moment and steel values are approximately correct. An error of any magnitude in the steel area is immediately shown by this check. Thus, if the steel area had been figured 15.34 sq. in., j would have had a value of 0.84. If the steel area had been figured as 13.34 sq. in., j would have had a value of 0.965.

Practical and economical work always holds to a minimum number of beam sizes, and variations in moment are met by variations in the quantity of reinforcement. A certain section may require adaptation to 10 or 12 different span and loading conditions. Therefore to have control sections carefully worked out and checked permits of very simple adjustments in other cases.

Having once found the capacity moment and steel, then for any lesser moment the reinforcement may be figured as proportional to the moments. What slight error is involved in this assumption is on the safe side.

(1) For the moment of 5,140,000 in. lb.

$$A_s = \frac{(5{,}140{,}000)}{(7{,}200{,}000)}\ (14.34 \text{ sq. in.}) = 10.22 \text{ sq. in.}$$

(2) For the moment 8,350,000 in. lb., compression steel must carry 8,350,000 in. lb. $-$ 7,200,000 in. lb. $=$ 1,150,000 in lb.

Locate and center the compressive steel 2 in. below top. Then

$$f_{cs} = 880 - \frac{(2)}{(28)}\ (1333 + 880) = 720 \text{ lb. per sq. in.}$$

$(n - 1)\ (f_{cs}) = (15 - 1)\ (720) = 10{,}080$ lb. per sq. in.
Lever arm $= 28 - 2 = 26$ in.

Therefore 1 sq. in. compressive steel has a moment value of

$$(10{,}080)\ (26) = 262{,}000 \text{ in. lb.}$$

Total area compressive steel required equals

$$\frac{1{,}150{,}000}{262{,}000} = 4.39 \text{ sq. in.}$$

Additional tension steel $\frac{(10{,}080)}{(20{,}000)}$ $(4.39) = 2.22$ sq. in.

Total tension steel 14.34 sq. in. + 2.22 sq. in. = 16.56 sq. in.

Shear and bond calculations are simple and standardized and need no discussion.

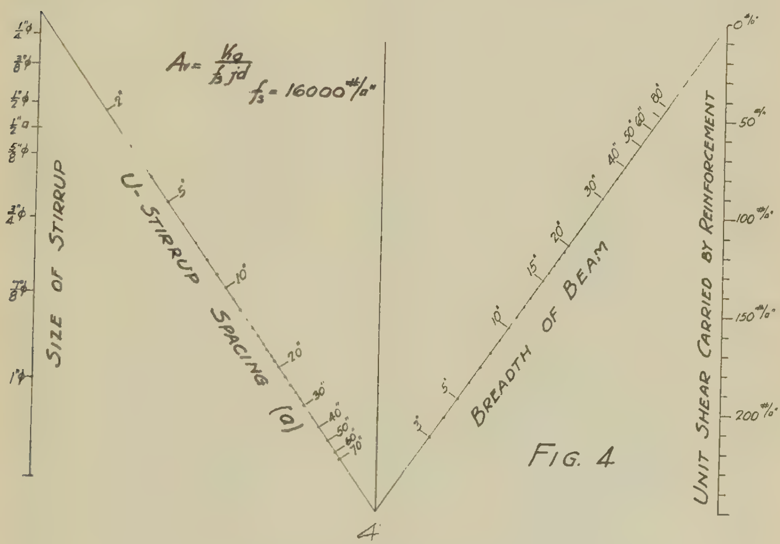


FIG. 4.—NOMOGRAPH FOR STIRRUP SPACING.

For the spacing of stirrups a simple and easy nomograph is submitted in Fig. 4. The unit shearing stress to be carried by stirrups is marked on the right hand vertical. By laying a straight edge between a point on this vertical and the proper point on the adjacent diagonal on which the breadths of beam stems are laid off, a point on the center vertical is obtained. Holding this point on the center vertical, the straight edge is now shifted and is swung from this point to intersect the vertical and diagonal at the left, from which are read simultaneous values of diameter or size of U-stirrups and the corresponding required spacing.

Such calculation of stirrup spacing is extremely quick, rapid, and likewise accurate and may be repeatedly made in less time than is involved

DESIGN VALUES FOR K AND p FOR DIFFERENT VALUES OF f_c

f_s = 20,000 lbs. per sq. in.

f_c	$n = 8$		$n = 10$		$n = 12$		$n = 15$	
	K	p	K	p	K	p	K	p
100.....	1.9	0.00010	2.3	0.00012	2.8	0.00014	3.4	0.00017
110.....	2.3	0.00012	2.8	0.00014	3.3	0.00017	4.1	0.00021
120.....	2.7	0.00014	3.3	0.00017	3.9	0.00021	4.8	0.00025
130.....	3.2	0.00016	3.9	0.00020	4.6	0.00024	5.6	0.00029
140.....	3.6	0.00019	4.5	0.00023	5.3	0.00027	6.4	0.00033
150.....	4.2	0.00021	5.1	0.00026	6.0	0.00031	7.3	0.00038
160.....	4.7	0.00024	5.8	0.00030	6.8	0.00035	8.3	0.00043
170.....	5.3	0.00027	6.5	0.00033	7.6	0.00039	9.2	0.00048
180.....	5.9	0.00030	7.2	0.00037	8.5	0.00044	10.3	0.00054
190.....	6.6	0.00034	8.0	0.00041	9.4	0.00049	11.4	0.00059
200.....	7.2	0.00037	8.8	0.00045	10.3	0.00054	12.5	0.00065
210.....	8.0	0.00041	9.7	0.00050	11.3	0.00059	13.6	0.00071
220.....	8.6	0.00044	10.5	0.00055	12.3	0.00064	14.8	0.00078
230.....	9.4	0.00049	11.5	0.00059	13.4	0.00070	16.1	0.00085
240.....	10.2	0.00053	12.4	0.00064	14.5	0.00076	17.4	0.00092
250.....	11.0	0.00057	13.4	0.00069	15.6	0.00082	18.7	0.00099
260.....	11.9	0.00061	14.4	0.00075	16.8	0.00088	20.1	0.00106
270.....	12.7	0.00066	15.4	0.00080	17.9	0.00094	21.5	0.00114
280.....	13.6	0.00070	16.5	0.00086	19.2	0.00101	22.9	0.00121
290.....	14.5	0.00075	17.6	0.00092	20.4	0.00107	24.4	0.00130
300.....	15.5	0.00080	18.7	0.00098	21.7	0.00114	25.9	0.00138
310.....	16.5	0.00086	19.9	0.00104	23.0	0.00122	27.4	0.00146
320.....	17.5	0.00091	21.1	0.00110	24.4	0.00129	29.0	0.00155
330.....	18.5	0.00096	22.3	0.00117	25.8	0.00136	30.6	0.00164
340.....	19.5	0.00102	23.5	0.00124	27.2	0.00144	32.2	0.00173
350.....	20.6	0.00108	24.8	0.00130	28.6	0.00152	33.9	0.00182
360.....	21.7	0.00113	26.1	0.00137	30.1	0.00160	35.4	0.00191
370.....	22.9	0.00120	27.4	0.00144	31.6	0.00168	37.3	0.00201
380.....	24.0	0.00125	28.7	0.00152	33.1	0.00176	39.0	0.00211
390.....	25.1	0.00132	30.1	0.00159	34.6	0.00185	40.8	0.00221
400.....	26.3	0.00138	31.5	0.00167	36.2	0.00194	42.6	0.00231
410.....	27.5	0.00144	32.9	0.00174	37.8	0.00202	44.4	0.00241
420.....	28.8	0.00151	34.3	0.00182	39.4	0.00211	46.3	0.00252
430.....	30.0	0.00158	35.8	0.00190	41.1	0.00220	48.2	0.00262
440.....	31.2	0.00164	37.3	0.00198	42.7	0.00230	50.1	0.00273
450.....	32.6	0.00172	38.8	0.00207	44.4	0.00239	52.0	0.00284
460.....	33.8	0.00178	40.3	0.00215	46.2	0.00249	54.0	0.00295
470.....	35.0	0.00185	41.9	0.00224	47.9	0.00259	55.9	0.00306
480.....	36.4	0.00193	43.4	0.00232	49.7	0.00268	57.9	0.00318
490.....	37.8	0.00200	45.0	0.00241	51.4	0.00278	59.9	0.00329
500.....	39.3	0.00208	46.7	0.00250	53.3	0.00288	62.0	0.00341
510.....	40.8	0.00216	48.3	0.00259	55.1	0.00299	64.0	0.00353
520.....	42.1	0.00223	50.0	0.00268	56.9	0.00309	66.1	0.00365
530.....	43.5	0.00231	51.6	0.00278	58.8	0.00320	68.2	0.00377
540.....	45.0	0.00239	53.3	0.00287	60.7	0.00330	70.4	0.00389
550.....	46.4	0.00247	55.0	0.00297	62.6	0.00341	72.5	0.00402
560.....	48.1	0.00256	56.8	0.00306	64.5	0.00352	74.7	0.00414

Values of "K" and "p" (continued)

f_c	$n = 8$		$n = 10$		$n = 12$		$n = 15$	
	K	p	K	p	K	p	K	p
570.....	49.5	0.00264	58.5	0.00316	66.5	0.00363	76.8	0.00427
580.....	51.2	0.00273	60.3	0.00326	68.4	0.00374	79.0	0.00440
590.....	52.6	0.00281	62.1	0.00336	70.4	0.00386	81.2	0.00452
600.....	54.3	0.00290	63.9	0.00346	72.4	0.00397	83.5	0.00466
610.....	55.9	0.00299	65.7	0.00356	74.4	0.00408	85.8	0.00479
620.....	57.5	0.00308	67.6	0.00367	76.5	0.00420	88.0	0.00492
630.....	59.1	0.00317	69.4	0.00377	78.5	0.00432	90.3	0.00505
640.....	60.8	0.00326	71.3	0.00388	80.6	0.00444	92.6	0.00519
650.....	62.4	0.00335	73.2	0.00399	82.7	0.00456	94.9	0.00533
660.....	64.2	0.00345	75.1	0.00409	84.7	0.00468	97.2	0.00546
670.....	65.8	0.00354	77.0	0.00420	86.8	0.00480	99.5	0.00560
680.....	67.6	0.00363	78.9	0.00431	89.0	0.00493	101.9	0.00574
690.....	69.2	0.00373	80.9	0.00442	91.2	0.00505	104.3	0.00588
700.....	71.0	0.00383	82.9	0.00454	93.3	0.00518	106.7	0.00602
710.....	72.6	0.00392	84.9	0.00465	95.5	0.00530	109.1	0.00617
720.....	74.4	0.00402	86.9	0.00476	97.7	0.00543	111.5	0.00631
730.....	76.2	0.00412	88.9	0.00488	99.9	0.00556	113.9	0.00646
740.....	78.0	0.00422	90.9	0.00500	102.1	0.00569	116.3	0.00660
750.....	79.8	0.00432	93.0	0.00511	104.4	0.00582	118.8	0.00675
760.....	81.5	0.00442	94.8	0.00522	106.3	0.00594	120.9	0.00688
770.....	83.4	0.00452	97.1	0.00535	108.8	0.00608	123.7	0.00705
780.....	85.3	0.00463	99.1	0.00547	111.1	0.00622	126.2	0.00720
790.....	87.2	0.00474	101.3	0.00560	113.4	0.00635	128.7	0.00735
800.....	89.1	0.00485	103.4	0.00571	115.7	0.00649	131.3	0.00750
810.....	91.1	0.00496	105.5	0.00584	118.0	0.00662	133.8	0.00765
820.....	92.9	0.00506	107.8	0.00597	120.3	0.00676	136.3	0.00781
830.....	94.8	0.00517	109.8	0.00609	122.7	0.00690	138.9	0.00796
840.....	96.7	0.00528	111.9	0.00621	125.0	0.00704	141.4	0.00812
850.....	98.6	0.00539	114.2	0.00634	127.4	0.00718	144.0	0.00827
860.....	100.7	0.00550	116.4	0.00647	129.7	0.00732	146.6	0.00843
870.....	102.7	0.00562	118.6	0.00659	132.1	0.00746	149.2	0.00859
880.....	104.6	0.00573	120.7	0.00672	134.5	0.00760	151.8	0.00875
890.....	106.6	0.00584	122.9	0.00685	136.9	0.00774	154.4	0.00891
900.....	108.7	0.00596	125.3	0.00698	139.4	0.00789	157.0	0.00907
910.....	110.6	0.00607	127.5	0.00711	141.8	0.00803	159.6	0.00923
920.....	112.7	0.00619	129.7	0.00725	144.2	0.00818	162.3	0.00939
930.....	114.7	0.00630	132.0	0.00738	146.7	0.00833	164.9	0.00955
940.....	116.7	0.00642	134.2	0.00751	148.9	0.00846	167.3	0.00970
950.....	118.9	0.00654	136.6	0.00765	151.5	0.00862	170.2	0.00988
960.....	121.0	0.00666	138.9	0.00778	154.1	0.00877	172.9	0.01005
970.....	123.0	0.00678	141.1	0.00792	156.5	0.00892	175.6	0.01021
980.....	125.0	0.00690	143.3	0.00806	159.1	0.00907	178.3	0.01038
990.....	127.2	0.00702	145.9	0.00819	161.6	0.00923	181.0	0.01055
1000.....	129.2	0.00714	148.2	0.00833	164.1	0.00938	183.7	0.01071
1010.....	131.4	0.00727	150.5	0.00847	166.6	0.00953	186.4	0.01088
1020.....	133.5	0.00739	152.8	0.00861	169.1	0.00968	189.1	0.01105
1030.....	135.6	0.00751	155.3	0.00875	171.7	0.00984	191.9	0.01122

Values of "K" and "p" (continued)

f_c	$n = 8$		$n = 10$		$n = 12$		$n = 15$	
	K	p	K	p	K	p	K	p
1040.....	137.8	0.00764	157.6	0.00889	174.2	0.00999	194.6	0.01139
1050.....	139.8	0.00776	160.0	0.00904	176.7	0.01014	197.8	0.01159
1060.....	142.1	0.00789	162.5	0.00918	179.3	0.01030	200.1	0.01174
1070.....	144.4	0.00802	164.8	0.00932	181.8	0.01045	202.9	0.01191
1080.....	146.5	0.00815	167.2	0.00947	184.4	0.01061	205.6	0.01208
1090.....	148.7	0.00827	169.6	0.00961	187.0	0.01077	208.3	0.01226
1100.....	150.9	0.00840	172.1	0.00976	189.6	0.01093	211.2	0.01243
1110.....	153.2	0.00853	174.5	0.00990	192.2	0.01109	214.0	0.01261
1120.....	155.4	0.00866	176.9	0.01005	194.9	0.01125	216.8	0.01278
1130.....	157.5	0.00879	179.5	0.01020	197.5	0.01141	219.6	0.01296
1140.....	160.0	0.00893	181.9	0.01035	200.1	0.01157	222.4	0.01314
1150.....	162.2	0.00906	184.3	0.01050	202.8	0.01174	225.2	0.01331
1160.....	164.3	0.00919	187.0	0.01065	205.4	0.01190	228.0	0.01349
1170.....	166.6	0.00932	189.4	0.01080	208.0	0.01206	230.8	0.01367
1180.....	169.0	0.00946	191.8	0.01095	210.8	0.01223	233.6	0.01385
1190.....	171.3	0.00959	194.4	0.01110	213.4	0.01239	236.4	0.01403
1200.....	173.6	0.00973	196.9	0.01125	216.2	0.01256	239.3	0.01421
1210.....	175.9	0.00987	199.3	0.01140	218.7	0.01272	242.2	0.01439
1220.....	178.3	0.01000	202.0	0.01156	221.5	0.01289	245.0	0.01457
1230.....	180.5	0.01014	204.5	0.01171	224.2	0.01306	247.9	0.01476
1240.....	183.0	0.01028	206.9	0.01186	226.8	0.01322	250.8	0.01494
1250.....	185.3	0.01042	209.6	0.01202	229.5	0.01339	253.6	0.01512
1260.....	187.6	0.01056	212.1	0.01217	232.3	0.01356	256.5	0.01530
1270.....	190.0	0.01070	214.6	0.01233	235.0	0.01373	259.4	0.01549
1280.....	192.3	0.01084	217.3	0.01249	237.8	0.01390	262.3	0.01567
1290.....	194.8	0.01098	220.6	0.01265	240.5	0.01407	265.2	0.01586
1300.....	197.1	0.01112	222.5	0.01280	243.2	0.01424	.....	....
1310.....	199.3	0.01126	225.0	0.01296	245.9	0.01441	.....	....
1320.....	201.8	0.01140	227.5	0.01312	248.6	0.01458	.....	....
1330.....	204.2	0.01155	230.3	0.01328	251.4	0.01475	.....	....
1340.....	206.7	0.01169	232.8	0.01344	254.2	0.01493	.....	....
1350.....	208.9	0.01183	235.6	0.01360	256.9	0.01510	.....	....
1360.....	211.6	0.01198	238.1	0.01376	.....	....	.....	....
1370.....	214.0	0.01213	240.5	0.01392	.....	....	.....	....
1380.....	216.2	0.01227	243.5	0.01409	.....	....	.....	....
1390.....	218.8	0.01242	246.0	0.01425	.....	....	.....	....
1400.....	221.1	0.01256	248.7	0.01441	.....	....	.....	....
1410.....	223.7	0.01271	.....	....	.....	....	.....	....
1420.....	226.1	0.01286	.....	....	.....	....	.....	....
1430.....	228.7	0.01301	.....	....	.....	....	.....	....
1440.....	231.1	0.01316	.....	....	.....	....	.....	....
1450.....	233.7	0.01331	.....	....	.....	....	.....	....
1460.....	236.1	0.01346	.....	....	.....	....	.....	....
1470.....	238.7	0.01361	.....	....	.....	....	.....	....
1480.....	241.1	0.01376	.....	....	.....	....	.....	....
1490.....	243.7	0.01391	.....	....	.....	....	.....	....
1500.....	246.1	0.01406	.....	....	.....	....	.....	....

in a single ordinary slide rule calculation. Two or three checks on the stirrup spacing between the point of maximum shear and that at which no stirrups are required will be sufficient in the majority of cases. We recognize the possibilities for precision that close adherence to a code will permit, but regard the subject of web stresses in concrete beams, or beams of any other material, as being altogether too vague and indefinite to justify refined mathematical calculation. When beams to which the 1924 Joint Committee Report would allow a working stress of 90 lb. per sq. in. in shear, carry up to 850 lb. per sq. in. in shear in the Seattle city testing laboratory, and then fail by bond, great care and exactitude in stirrup spacing does not appear to us to be warranted.

For the case where bent up bars occur and furnish the best possible web reinforcement, the writers use another simple nomograph in which the values of the bent up bars are expressed in terms of the vertical stirrups they replace.

In conclusion we would express the hope that methods of design of reinforced concrete may be so simplified in the future that the work involved in securing the most accurate and economical designs may be reduced to a minimum.

ARTHUR R. LORD.*—The alternate design method of Messrs. Hadley and Way is one of great value and possesses the high merit of being quite simple in use without sacrificing great mathematical refinement. The possibilities of the method are particularly brought out by their Example 2 where the two flanges of a T-beam are markedly dissimilar and where some sort of cumulative design method must be applied even when the tables in my paper are used. **Mr. Lord**

The discussion is most interesting also in illustrating to perfection the perpetual battle that every structural designer must wage between "practical" and "theoretical" considerations. We have designed many buildings using stem widths for beams of 9⅝ in., 11⅝ in., 13¼ in., etc., instead of 10 in., 12 in., 14 in., etc., in order to avoid the need of ripping lumber for beam bottoms; and we have found the average contractor surprisingly unimpressed by such aid from the engineer. I have called attention in the text (page 542, *Proceedings* A.C.I., 1928) to the matter of keeping beam sizes uniform for the sake of formwork economy. The point is well taken as every designer of experience must know.

Practical considerations are important in the office as well as in the field and in connection with the labor of computations no less than with that of construction. I will agree cheerfully that the use of the full available width of flange permitted to be used in the code is logical and represents more closely the actual condition as to distribution of compressive stress. But it is also laborious, while the resulting designs in some cases are identical and in other cases result in a slight increase in steel by my method to offset the saving in computation. It is obvious that the selection of values for K and p from a table for T-beams or doubly reinforced beams is simpler and quicker than the mathematical calculations required to obtain K and p by the Hadley and Way method.

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For the case illustrated by Example 1, the values by the two methods will be found to be identical at the support, if my tables be *interpolated* (as directed on page 550). For rectangular and T-beams the use of the full allowable flange width results in a saving in the reinforcement required, while in T-beams the inclusion in the computation of the web between the neutral axis and the bottom of the flange also results in a saving in some cases. These two savings are not enough to change the bar selection in the example, however, and it seems to me that the method given in my paper is more "practical" in the average case than the Hadley and Way method, even though not exact to the same decimal place. In the problems worked out in my paper I have not even interpolated but have used the nearest value direct from the table and this I feel to be the "practical" thing to do in design. After limiting beams have been determined the experienced designer will often be able to proceed directly from the moment to the steel area, using my formula (103b) as stated on page 542, and so effect a great saving in design labor as compared with my full method or with the Hadley and Way method. If formula (103b) were rewritten as formula (103c)

$$A_s = \frac{M}{20000\,jd} \qquad (103c)$$

it would give identical results with the Hadley and Way method. In (103b) j has been taken as ⅞ in conformity with the code and to reduce labor in design. For the case of typical beams, in which even the slightest saving is multiplied by duplication of members, I believe that the Hadley and Way method is an excellent one to use. As stated in my paper, I would advise a designer to *interpolate* in my tables for such cases.

The "practical" vs. the "theoretical" issue comes out repeatedly in this discussion. As to stirrups, Messrs. Hadley and Way seem to me to repose an unwarranted faith in some beams tested at the Seattle laboratory. This is no reflection on that laboratory but it does seem to me that hundreds of other tests made in many laboratories should have equal weight, especially when they indicate very much *lower* shearing resistances. I personally prefer to show a theoretically accurate stirrup spacing to the nearest inch on my drawings, and I have the utmost confidence that the men on the job will impart all needed artistic irregularities to the finished work. Whereas my web reinforcement diagrams give in one simple operation the theoretical stirrup spacing for *all stirrups at one end* of a beam, the Hadley and Way method requires as many operations as there are stirrups and is vastly more laborious if accurate results are desired. While the final use of their Fig. 4 is rapid, each such use requires considerable computation (to find v_s) as a preliminary.

Messrs. Hadley and Way appear also to be on the "practical" side with respect to the choice of the proper value of n—but are they? They want to be able to design for 2275-lb. concrete, while I am satisfied to call it 2000 or 2500-lb. concrete and feel content if the cylinders test from 1 to 15 per cent above my assumed strength. In any case my paper

is based on the Joint Code which sets up a sliding value of n and I could not change it if I would (and would not if I could, because with a sliding value of n I can interpolate readily). It may seem strange to use $n = 13.6$ for an 2200-lb. concrete but is it any stranger than to contemplate the wreckage to your computations if you had used $n = 12$ and had the misfortune to have your concrete strength drop off to 2199 lb. per sq. in. for which n becomes 15!

Of course this sort of discussion is completely academic. My own feeling is that the "practical" requirement will be fully met by the use of 2000, 2500, 3000 and 3750-lb. concrete and that we will gain more by standardizing on a few strengths and learning how to secure them on the job than we would by having each designer fancy free to specify any strength that may come to his mind. I have had enough contracting experience to believe that contractors would prefer to fix on a few strengths rather than on a few values of n, with which symbol they are not generally very familiar. The use of 2000, 2500, 3000 and 3750-lb. concrete is also consistent with the value of $j = 7/8$ used throughout the code (as a "practical" matter doubtless). The Hadley and Way method involves the use of all theoretical values of j, and affects the steel savings mentioned (principally) by using high values of j. I do not feel that this is at all wrong, even though it appears to be a technical violation of the code. It is sanctioned to a degree by the words "accepted theory of flexure" in section 601 (a) of the code.

At this point I believe it would be well to insert a word of warning about the use of Table 1 of Hadley and Way. It is entirely proper, within sensible bounds, to use a fiber stress of 100 or 200 lb. per sq. in. with the value of n taken as 8, 10, 12 or 15. If the fiber stress used in the design of the extreme compression fibers of the beam is 800 lb. per sq. in. (or thereabouts), corresponding to the usual 2000-lb. concrete, then n may be properly taken as 15 for all fibers intermediate between the extreme fiber and the neutral axis, as has been done in this discussion. I would consider it very doubtful, however, to use in a member an extreme fiber stress as low as 100 or 200 lb. per sq. in. with n taken as 15, where it should probably be taken much lower. Table 1 lends itself to abuse in this respect, as it does also in giving values for fiber stresses as high as 1290 lb. per sq. in. with $n = 15$. Assuming the usual stress-strain relationship exhibited by concrete, the value of n should be much larger for this case, the more so because the phenomenon of "plastic flow" will be much more marked at such high stresses. On the other hand for $n = 8$ only, I would suggest that Table 1 be extended to include fiber stresses up to 1690 lb. per sq. in. ($0.45\,f'_c$).

Confusion may result also from the discussion given of my Problem 2 illustrating the design of an ordinary T-beam. Messrs. Hadley and Way apparently reach the conclusion that a beam having a flange width of 72 in. and a stem width of 12 in. (as used in their Example 1) is a *rectangular* beam rather than a T-beam. The choice of terms is unfortunate. By using the full available width of T-flange, this beam, in accordance

with American Standard notation, has been changed from a *Case II* T-beam into a *Case I* T-beam, but it obviously must be treated as a T-beam in design since any other treatment would lead to great error in the web reinforcement.

In conclusion may I say that I think sufficiently well of this alternate method that I shall ask the authors' permission to include it with proper credit in a second edition of my tables and diagrams, should such be printed. I wish also to state that, as a direct result of this discussion, I modified tables 8 to 19 inclusive, between the preprint and the final printing, to reduce the size of the steps between the tabulated values and so to reduce the need of interpolation when using these tables in the manner illustrated by the problems. No change in mathematical treatment or in accuracy was involved. A slight change in the numerical values in my Problem 2, made at the same time, is also responsible largely for the great difference in steel areas shown. Using corrected figures A'_s becomes 4.09 sq. in. (instead of 3.68 sq. in) as compared with my 4.32 sq. in.

Mr. Hadley and Mr. Way.

H. M. HADLEY and W. F. WAY—Without discussing points that are merely debatable, and without wishing to appear unappreciative of Mr. Lord's words of commendation, the writers of the preceding discussion desire to make a few additional statements, in view of Mr. Lord's rebuttal.

Our opinions regarding web stresses and stirrup spacing are based not solely upon Seattle tests but upon the variability of "hundreds of other tests" to which Mr. Lord refers. Mr. Richart's frame tests, reported in the Bureau of Standards *Journal of Research*, August, 1928, show shearing stresses ranging up to 1600 lb. per sq. in. It is not the magnitude but the variability of these results that impresses us. Therefore we are of the opinion that in the ordinary case, and observing the standard rules regarding maximum spacing, quite as much accuracy is obtained by one or two changes in stirrup spacing as by changing the spacing with every stirrup, e.g., we believe that "4 at 8 in., 3 at 12 in.," will more simply and quite as accurately and effectively reinforce the web of a beam as "7 in., 8 in., 9 in., 10 in., 11 in., 12 in., 12 in.," and appear much more "practical" to the man on the job.

As an illustration of the flexibility of the tables we stated that they were applicable to a 2275-lb. concrete. We did not state that we wanted to be able to design with such a concrete.

In view of the effect of water and of a few other things upon concrete strength, it is not apparent to the writers how or why we should standardize upon and ordain 4 certain concretes, which happen to be the only concretes having values of n under the proposed Joint Code formula.

While j has all theoretical values in our tables, these are all happily concealed and come forth to trouble no one, either the hardy practical contractor or the refined theorist.

Messrs. Hadley and Way stated that the beam of Example 1 was "classed as a rectangular beam" and was to be "figured as a rectangular beam." They did not reach the conclusion that it *was* a rectangular beam.

They do not understand what Mr. Lord means by saying, "any other treatment would lead to great error in the web reinforcement."

There is a further matter and a truly practical one which should be specifically mentioned, although perhaps it is obvious. It is this: In those cases where building codes, or differences of engineering opinion and judgment do not permit the use of design working stresses of 0.40 and 0.45 of the ultimate concrete strength, our tables are in nowise the less serviceable and adaptable. Whatever fraction or percentage of the ultimate strength may be selected as the working stress, these tables can be used. Mr. Lord's tables, under such conditions, cannot be used. They cannot, by interpolation or otherwise, be used in Seattle today, because the Seattle building code permits as maximum working stresses, not 0.40 and 0.45 of the ultimate concrete strength, but 0.375 and 0.41 of the ultimate concrete strength. There are other places where Mr. Lord's tables cannot be used. It should not be forgotten that adaptability to use is an aspect of practicality.

The foregoing is not to be construed as a reflection upon Mr. Lord's work. His work was to prepare design tables and data in conformity with the proposed 1928 Joint Standard Building Code. This is precisely what he has done. But in doing this, his results are not adaptable to use with other codes and specifications. Our tables are conformable to the proposed 1928 Joint Standard Building Code—or to any other code permitting a working stress of 20,000 lb. per sq. in. in the reinforcement, and as we stated, with our similar tables based on 16,000 lb. and 18,000 lb. working stress, cover practically the entire range of codes and specifications. There is this "practical" difference between the two methods.

JOSEPH A. KITTS—The author of this data for structural design has contributed a work of great value to the general welfare. Savings of $0.05 to $0.25 per total square footage of floor area may be expected and more durable buildings will result. **Mr. Kitts.**

It is well known to concrete physicists that concrete of 3000-4000 lb. strength can be made more economically in proportion to its strength than 2000-lb. concrete, the ordinary basis of design. Use of this fact by the author explains why it is possible to design more economically with the usual safety factor.

Recent developments, formulating a concrete technology, make it practicable to produce 4000-lb. concrete with greater exactness and uniformity than 2000-lb. concrete has been produced heretofore, and 3000-4000 lb. concrete, so produced, has the following advantage over 2000-lb. concrete:

(1) Greater strength in proportion to cost;

(2) Better workability, flowability and cohesion in the fresh mix and economy in placing;

(3) Optimum conditions of strength, density, impermeability and elasticity;

(4) Minimum shrinkage in setting and plastic flow under stress;

(5) Greater durability to wear and exposure; and

(6) Resulting economy.

The following tables, comprising analyses of data published by the Portland Cement Association under the title "Design and Control of Concrete Mixtures" (January, 1927), show that concrete of greatest economy is that having (*a*) Strength of 3000 to 4000 lb. per sq. in.; (*b*) maximum usable size of aggregate; (*c*) minimum usable water-cement ratio; and (*d*) skilled control of production.

Our experience shows that these data are substantially correct under average conditions with normal standard portland cement and aggregate of standard quality. They are offered as supplementary cost data for the purpose of comparative design.

Cost of cement is assumed as $3.00 per barrel and aggregate at $2.00 per ton delivered to the mixer.

EXPLANATION OF TABLES 1 TO 4

The tables show for concretes of various 28-day strengths: (1) The true mix; (2) sacks of cement per cu. yd. of concrete; (3) cost of materials per cu. yd. of concrete; and (4) compressive strength in proportion to cost of materials.

Also these data are given for both (*a*) skilled control; and (*b*) ordinary control; and for ¾, 1, 1½, 2 and 3 in. (square hole) maximum size of aggregate for both cases (*a*) and (*b*).

Each table is for a particular consistency of mix as indicated by the standard slump test.

TABLE 1—CONCRETE WITH ½ TO 1-IN. SLUMP

(Suitable for plain concrete pavements and plain mass concrete only in situations where flowability is not essential and particularly for aggregate over 2 in. maximum.)

Strength at 28 Days in lb. per sq. in.	Skilled Control — Maximum Size of Aggregate					Ordinary Control — Maximum Size of Aggregate				
	¾ in.	1 in.	1½ in.	2 in.	3 in.	¾ in.	1 in.	1½ in.	2 in.	3 in.
TRUE MIX—VOLUME OF DRY RODDED MIXED AGGREGATE PER SACK OF CEMENT										
1500	7.70	8.35	9.20	9.85	10.00	6.65	7.25	7.85	8.50	9.45
2000	6.55	7.15	7.65	8.35	9.15	5.45	5.90	6.45	7.00	7.70
2500	5.45	5.90	6.45	6.95	7.65	4.55	4.90	5.35	5.80	6.35
3000	4.65	4.95	5.40	5.90	6.40	3.75	4.10	4.45	4.85	5.30
3500	3.85	4.20	4.55	4.95	5.40	3.10	3.30	3.65	3.95	4.30
4000	3.15	3.40	3.70	4.15	4.45	2.45	2.65	2.90	3.20	3.45
SACKS OF CEMENT PER CU. YD. OF CONCRETE										
1500	3.40	3.10	2.84	2.65	2.60	3.95	3.60	3.35	3.05	2.77
2000	4.02	3.65	3.42	3.10	2.87	4.83	4.45	4.08	3.70	3.40
2500	4.83	4.45	4.08	3.75	3.42	5.65	5.30	4.90	4.55	4.12
3000	5.55	5.25	4.87	4.45	4.10	6.65	6.20	5.80	5.35	4.95
3500	6.50	6.05	5.65	5.25	4.87	7.70	7.35	6.90	6.40	5.95
4000	7.60	7.15	6.75	6.10	5.80	9.30	8.70	8.10	7.50	7.10
COST OF MATERIALS PER CU. YD. OF CONCRETE										
1500	6.06	5.92	5.82	5.77	5.66	6.38	6.20	6.06	5.97	5.82
2000	6.48	6.32	6.19	6.03	5.88	6.88	6.68	6.51	6.31	6.12
2500	6.94	6.77	6.59	6.43	6.25	7.43	7.22	7.00	6.80	6.58
3000	7.43	7.25	7.06	6.84	6.66	8.04	7.82	7.59	7.33	7.11
3500	8.00	7.79	7.58	7.35	7.15	8.74	8.49	8.24	7.97	7.73
4000	8.70	8.47	8.22	7.88	7.70	9.62	9.35	9.06	8.68	8.46
ECONOMY FACTOR—COMPRESSIVE STRENGTH (LB. PER SQ. IN.) PER DOLLAR COST OF MATERIALS										
1500	248	253	258	260	265	235	242	248	251	258
2000	309	316	323	332	340	291	299	307	317	327
2500	360	369	379	389	400	336	346	357	368	380
3000	404	414	425	439	450	373	384	395	409	422
3500	438	449	462	476	490	400	412	425	439	453
4000	460	472	487	508	519	416	428	442	461	473

TABLE 2—CONCRETE WITH 3 TO 4-IN. SLUMP

(Suitable for concrete pavements and for mass concrete only where flowability is not essential and particularly for aggregate over 1½ in. maximum.)

Strength at 28 Days in lb. per sq. in	Skilled Control					Ordinary Control				
	Maximum Size of Aggregate					Maximum Size of Aggregate				
	¾ in.	1 in.	1½ in.	2 in.	3 in.	¾ in.	1 in.	1½ in.	2 in.	3 in.
TRUE MIX—VOLUME OF DRY RODDED MIXED AGGREGATE PER SACK OF CEMENT										
1500	6.80	7.45	8.10	8.80	9.65	5.85	6.40	6.90	7.50	8.25
2000	5.65	6.25	6.75	7.35	7.95	4.70	5.15	5.60	6.10	6.65
2500	4.65	5.15	5.55	6.10	6.55	3.90	4.25	4.65	5.00	5.45
3000	3.85	4.25	4.65	5.15	5.55	3.15	3.40	3.75	4.10	4.45
3500	3.25	3.50	3.85	4.20	4.50	2.55	2.75	3.05	3.30	3.55
4000	2.60	2.80	3.10	3.35	3.65	1.95	2.15	2.35	2.60	2.80
SACKS OF CEMENT PER CU. YD. OF CONCRETE										
1500	3.85	3.50	3.22	2.95	2.70	4.50	4.10	3.80	3.48	3.15
2000	4.65	4.20	3.90	3.55	3.30	5.50	5.10	4.70	4.30	3.95
2500	5.55	5.10	4.75	4.30	4.02	6.45	6.00	5.55	5.20	4.83
3000	6.50	6.00	5.55	5.10	4.75	7.60	7.15	6.65	6.20	5.80
3500	7.40	7.00	6.50	6.05	5.75	9.00	8.50	7.90	7.35	6.95
4000	8.85	8.35	7.70	7.25	6.90	11.20	10.40	9.70	8.85	8.35
COST OF MATERIALS PER CU. YD. OF CONCRETE										
1500	6.26	6.10	5.99	5.87	5.75	6.66	6.45	6.29	6.12	5.96
2000	6.78	6.59	6.44	6.28	6.15	7.25	7.01	6.81	6.60	6.43
2500	7.30	7.09	6.91	6.73	6.57	7.86	7.60	7.37	7.14	6.94
3000	7.90	7.67	7.44	7.22	7.04	8.59	8.31	8.03	7.76	7.54
3500	8.53	8.31	8.03	7.78	7.60	9.41	9.14	8.81	8.51	8.29
4000	9.53	9.19	8.83	8.48	8.27	10.67	10.28	9.87	9.47	9.22
ECONOMY FACTOR—COMPRESSIVE STRENGTH (LB. PER SQ. IN.) PER DOLLAR COST OF MATERIALS										
1500	240	246	250	256	261	225	233	238	245	252
2000	295	303	311	318	325	276	285	294	303	311
2500	342	353	362	371	381	318	329	339	350	360
3000	380	391	403	416	426	349	361	374	387	398
3500	410	421	436	450	461	372	383	397	411	422
4000	420	435	453	472	484	375	389	405	422	434

TABLE 3—CONCRETE WITH 6 TO 7-IN. SLUMP

(Suitable for reinforced concrete where usual flowability is required and particularly for aggregate under 2 in. and over ¾ in. maximum.)

Strength at 28 Days in lb. per sq. in.	Skilled Control — Maximum Size of Aggregate					Ordinary Control — Maximum Size of Aggregate				
	¾ in.	1 in.	1½ in.	2 in.	3 in.	¾ in.	1 in.	1½ in.	2 in.	3 in.
TRUE MIX—VOLUME OF DRY RODDED MIXED AGGREGATE PER SACK OF CEMENT										
1500	5.70	6.25	6.75	7.30	8.15	4.80	5.20	5.65	6.15	6.70
2000	4.70	5.10	5.50	6.00	6.55	3.75	4.00	4.40	4.80	5.25
2500	3.75	4.05	4.50	4.85	5.30	3.00	3.25	3.55	3.90	4.20
3000	3.10	3.30	3.55	3.90	4.35	2.35	2.55	2.80	3.00	3.25
3500	2.50	2.65	2.85	3.15	3.45	1.80	1.95	2.15	2.30	2.50
4000	1.85	2.00	2.20	2.40	2.60	1.25	1.35	1.50	1.60	1.75
SACKS OF CEMENT PER CU. YD. OF CONCRETE										
1500	4.6	4.2	3.9	3.5	3.2	5.4	5.0	4.7	4.3	3.9
2000	5.5	5.1	4.8	4.4	4.0	6.7	6.3	5.8	5.4	5.0
2500	6.7	6.2	5.8	5.4	4.9	7.9	7.4	6.9	6.5	6.1
3000	7.7	7.4	6.9	6.5	5.9	9.6	9.0	8.4	7.9	7.5
3500	9.2	8.7	8.4	7.6	7.1	11.9	11.2	10.4	9.8	9.1
4000	11.7	11.0	10.1	9.5	8.8	15.2	14.6	13.7	13.1	12.2
COST OF MATERIALS PER CU. YD. OF CONCRETE										
1500	6.63	6.46	6.33	6.15	5.99	7.07	6.87	6.71	6.52	6.34
2000	7.28	7.09	6.91	6.71	6.51	7.80	7.58	7.37	7.16	6.94
2500	7.94	7.70	7.47	7.27	7.05	8.60	8.33	8.07	7.86	7.61
3000	8.64	8.42	8.14	7.91	7.65	9.56	9.31	9.00	8.76	8.48
3500	9.63	9.29	8.89	8.63	8.32	10.97	10.60	10.17	9.90	9.57
4000	11.00	10.64	10.15	9.83	9.40	13.05	12.67	12.15	11.82	11.37
ECONOMY FACTOR—COMPRESSIVE STRENGTH (LB. PER SQ. IN.) PER DOLLAR COST OF MATERIALS										
1500	226	232	237	244	250	212	218	224	230	236
2000	275	282	289	298	307	256	264	271	279	288
2500	315	325	335	344	355	291	300	310	318	329
3000	347	356	369	379	392	314	322	333	342	354
3500	363	377	394	406	421	319	330	344	354	366
4000	364	376	394	407	426	308	316	329	338	352

TABLE 4—CONCRETE WITH 8 TO 10-IN. SLUMP

(Suitable for reinforced and thin sections where the maximum flowability is required and particularly for aggregate under ¾-in. maximum.)

Strength at 28 Days in lb. per sq. in.	Skilled Control					Ordinary Control				
	Maximum Size of Aggregate					Maximum Size of Aggregate				
	¾ in.	1 in.	1½ in.	2 in.	3 in.	¾ in.	1 in.	1½ in.	2 in.	3 in.
TRUE MIX—VOLUME OF DRY RODDED MIXED AGGREGATE PER SACK OF CEMENT										
1500	4.35	4.70	5.20	5.65	6.15	3.40	3.75	4.10	4.45	4.85
2000	3.35	3.70	4.00	4.45	4.25	2.60	2.80	3.10	3.40	3.65
2500	2.70	2.85	3.15	3.40	3.76	1.96	2.10	2.30	2.45	2.70
3000	2.10	2.20	2.40	2.60	2.80	1.35	1.45	1.60	1.75	1.87
3500	1.50	1.58	1.67	1.85	2.10	0.85	9.90	1.00	1.10	1.25
4000	1.05	1.10	1.15	1.25	1.35	0.40	0.45	0.53	0.55	0.65
SACKS OF CEMENT PER CU. YD. OF CONCRETE										
1500	5.9	5.5	5.0	4.7	4.3	7.2	6.7	6.2	5.8	5.3
2000	7.3	6.7	6.3	5.8	5.3	8.9	8.4	7.7	7.2	6.8
2500	8.6	8.2	7.6	7.2	6.7	11.2	10.6	9.8	9.3	8.6
3000	10.6	10.2	9.5	8.9	8.4	14.5	13.9	13.0	12.1	11.6
3500	13.6	13.1	12.6	11.7	10.6	18.3	17.9	17.1	16.3	15.2
4000	16.7	16.3	16.0	15.2	14.5	21.8	21.4	20.7	20.7	19.8
COST OF MATERIALS PER CU. YD. OF CONCRETE										
1500	7.32	7.11	6.88	6.72	6.54	7.89	7.66	7.40	7.22	7.01
2000	8.13	7.86	7.60	7.35	7.14	8.96	8.67	8.38	8.11	7.87
2500	8.97	8.73	8.37	8.15	7.84	10.25	9.99	9.60	9.36	9.02
3000	10.26	10.01	9.59	9.20	8.94	12.14	11.87	11.42	11.01	10.72
3500	11.97	11.72	11.39	10.92	10.33	14.53	14.26	13.90	13.41	12.79
4000	13.63	13.42	13.15	12.93	12.50	16.94	16.71	16.41	16.17	15.71
ECONOMY FACTOR—COMPRESSIVE STRENGTH (LB. PER SQ. IN.) PER DOLLAR COST OF MATERIALS										
1500	205	211	218	223	229	190	196	203	208	214
2000	246	255	263	272	280	223	231	239	247	254
2500	279	286	299	307	319	244	250	260	267	277
3000	292	300	313	326	336	247	253	263	272	280
3500	292	299	307	321	339	241	245	252	261	274
4000	293	298	304	309	320	236	239	244	247	255

EXAMPLE

Assuming 1 in. maximum size of aggregate and 6 to 7 in. slump for 2000 and 3500 lb. concrete for skilled and ordinary control, we find, in Table 3, the following comparative figures for 1 cu. yd. of concrete:

TABLE 5—COMPARISON OF SKILLED AND ORDINARY CONTROL

	Skilled	Ordinary	Skilled	Ordinary
Strength at 28 days	2000	2000	3500	3500
Dry-rodded aggregate per sack	5.10	4.00	2.65	1.95
Sacks per cu. yd. of concrete	5.10	6.30	8.70	11.20
Cost of materials	$7.09	$7.58	$9.29	$10.60
Pounds strength per dollar	282	264	377	330
Saving by skilled control	$0.49	....	$1.31	....

The relative costs of skilled and ordinary control are as shown in Table 6.

TABLE 6—RELATIVE COSTS OF SKILLED AND ORDINARY CONTROL

Cubic Yards of Concrete in Project	Skilled Control Cost per cu. yd. of Concrete	Ordinary Control Cost per cu. yd. of Concrete
1,000	$0.43	$0.21
2,000	0.365	0.155
3,000	0.33	0.13
4,000	0.305	0.115
5,000	0.29	0.105
6,000	0.275	0.10
7,000	0.265	0.095
9,000	0.25	0.085
10,000	0.24	0.08
12,000	0.23	0.075
14,000	0.22	0.07
17,000	0.21	0.065
20,000	0.20	0.06
25,000	0.19	0.055
30,000	0.18	0.05
40,000	0.17	0.045
50,000	0.16	0.04
500,000	0.09	0.015
1,000,000	0.075	0.011

Assuming a 10,000-cu. yd. project, the excess cost of skilled control of the concrete production is $0.24 minus $0.08 or $0.16, as shown in Table 6. This excess cost is due to the fact that one man-hour of skilled control is required for every 10 cu. yd. of concrete as compared with one man-hour of ordinary control for every 23 cu. yd. The net saving by skilled control, however, with conditions as given for Table 5, would be 10,000 ($0.49 – $0.16) = $3,300 for 2000-lb. concrete, and 10,000 ($1.31 = $0.16) – $11,500 for 3500-lb. concrete.

Tables 1, 2, 3, 4, and 6 provide the structural designer with data for the determination of comparative costs of designs as affected by the strength of concrete used and by the character of concrete production control.

It is obvious, of course, that the use of higher strength concrete permits of lighter design for the same live loads and this reduction of the dead load correspondingly reduces the cost of provision for carrying the dead load.

AMERICAN CONCRETE INSTITUTE

BUSINESS REPORTS

Annual Report of the Board of Direction to the Members—May 1, 1929

With the completion of this the 25th annual volume of *Proceedings* the Institute has already under way a new schedule of publications.

Growth of membership, increased interest in the work of the society and a consequent increased demand for opportunity to discuss a great variety of technical subjects related to the development of concrete design, construction and manufacture have gradually resulted in the inadequacy of the annual convention and an annual volume of *Proceedings* to meet the Institute's opportunities for usefulness. The annual 3-day convention became the "bottle-neck" of the society's activity. Programs were congested and discussion too restricted.

Volume 26 will therefore be published periodically as *The Journal of the American Concrete Institute,* edited by the Secretary, under the direction of a Publications Committee.

Program and Publications Committees will cooperate in the consideration of papers and reports. The Program Committee will select material believed best adapted to the purposes of conventions which in general will aim to give more thorough consideration to fewer major subjects. Some contributions for the annual meeting will be published in the *Journal* in advance of their convention presentation (as, in former years in separate prints) with a view to building up discussion. Other contributions will be published in the *Journal* only after their convention presentation. Their disposition in this respect will be the responsibility of the Program Committee.

It is anticipated that the great bulk of papers and reports will be published in the *Journal* and their discussion confined to written contributions in subsequent *Journal* issues.

The *Journal* will embody three main parts:

(1) *The Proceedings,* paged continuously through a volume of ten numbers, to facilitate binding with index at the conclusion of the volume.

(Members anticipating that individual issues of the *Journal* may be battered or lost and desiring to insure receipt of a complete bound volume at the end of the year, may place with the Secretary advance orders to save complete sets of *Proceedings* pages and bind them at the end of the *Journal* year. Orders for such duplicate service will be accepted at their estimated cost.)

(2) Abstracts of important current literature on concrete will form a group of pages in each *Journal.* These will also be paged continuously through the year (distinct from *Proceedings*) to facilitate separate yearly binding.

(3) The *News Letter* will be continued, but as a part of the *Journal,* as a means of advising members of organization work and for the more casual exchange of ideas in the Institute's field.

Inevitably, technical committee activity has centered upon the annual convention. The inauguration of periodical publication necessitates activity more evenly distributed throughout the year. It also imposes new responsibilities of alertness to active or potential developments in the Institute's field of interest.

Individuals are generally more responsive to such a continuous program than are committees. In reorganizing the Institute's technical program the responsibilities of alertness to developments are divided among nine departments of interest—each department with a chairman and two co-chairmen. The nine chairmen are members of the Advisory Committee which recommends to the Board of Direction the organization or discharge of committees, the appointment of chairmen and to the president the appointment of committee members.

All former technical committees were discharged by action of the Board in February. About forty new committees have been or are being organized.

Some of these committees are essentially the same as former committees, continuing work which had been undertaken. Most of the new committees will have but four members—an Author-Chairman who will write the report asked for and three critics who will criticise it before publication. A majority of a committee must approve before a report becomes acceptable. Such committee operation frankly recognizes what in general has been true—that one man usually takes the initiative in writing a report and deserves recognition as the author. Most of these new committees have limited and definite assignments. When the assigned work is completed the committee will be discharged.

Thus, committees or small working units have definite tasks usually for prompt action and it is believed the result will contribute to the success of the new publication policy.

This volume is in itself a report of the Institute's work for the year —a year in which more papers and reports were preprinted (in advance of the 25th annual convention) than ever before and Institute literature had much wider and more general distribution.

The registration at the Detroit convention was 685.

June 30, 1927, we had 2 Honorary members, 120 Supporting members, 2193 Active members—a total membership of 2315. At the close of the last fiscal year, June 30, 1928: Honorary members 2; Supporting members 121; Active members 2407—a total membership of 2530. May 1, 1929, the total membership is 2738.

The report of the annual audit for the fiscal year ended June 30, 1928, is as follows:

MEISSNER AUDIT CO.
DETROIT, MICH.

August 1, 1928.

Mr. Harvey Whipple, Treasurer,
American Concrete Institute,
2970 West Grand Boulevard,
Detroit, Michigan.

DEAR SIR:

We have, pursuant to your request, conducted a cash audit of the American Concrete Institute, from July 1, 1927, to June 30, 1928, as shown by the books of the company.

All cash reported received was deposited in the National Bank of Commerce of Detroit, Mich.

Disbursements and withdrawals were covered by properly approved vouchers.

The bank account was audited and found to agree with statement of depository bank.

No attempt was made by us to verify cash receipts as reported, with the various members by communication.

The following schedules are attached hereto:

Balance sheet, as of June 30, 1928.

Receipts and Disbursements, July 1, 1927, to June 30, 1928.

Bank reconciliation, June 30, 1928.

Respectfully yours,

MEISSNER AUDIT COMPANY

By J. M. MEISSNER
Public Accountant

AMERICAN CONCRETE INSTITUTE
BALANCE SHEET

June 30, 1928

ASSETS

Cash:		
On hand	$500.00	
In National Bank of Commerce	7,273.17	
Total Cash		$7,773.17
Securities:		
U. S. Treasury Certificates	$8,133.13	
Oakland County Highway bonds	3,025.59	
Total Securities		11,158.72
Accounts Receivable:		
Active Members	$5,623.08	
Contributing Members	400.00	
Miscellaneous	1,180.17	
Total Accounts Receivable		7,203.25
Inventory, Proceedings		1,430.50
Total Assets		$27,565.64

LIABILITIES

Accounts Payable, Proceedings	$7,078.21	
Deferred:		
Dues paid in advance	238.75	
Reserve:		
For loss due to delinquent members	3,500.00	
Surplus:		
Deposited by members	16,748.68	
Total Liabilities		$27,565.64

AMERICAN CONCRETE INSTITUTE
RECEIPTS & DISBURSEMENTS

Year Ending June 30, 1928

Cash on hand, July 1, 1927		$4,241.04

RECEIPTS

Matured, Treasurer Certificates	$2,000.00	
Dues, Active	26,440.17	
Dues, Contributing	5,987.50	
Preprint sales	2,205.23	
Proceedings sales	1,326.60	
Interest earned on bank balances	117.57	
Interest earned on Treasury Certificates	323.20	
Overpaid in 1927 on Sterling check	11.61	
Certificates, Membership	2.48	
Exchanges, net	2.99	
Revenue for use of type	1,200.00	
Total Receipts		39,617.35
		$43,858.39

Disbursements

Auditing	$34.38	
Conventions	1,002.90	
Express and freight	42.88	
Membership, National Fire Protection Association	60.00	
Miscellaneous contingencies	153.05	
Office expense	774.63	
Postage	1,440.97	
Preprint expense	4,986.35	
Printing, multigraphing, stationery and News Letters	3,867.13	
Proceedings expense	7,791.76	
Rent	720.00	
Secretary's bond	50.00	
Salaries	10,144.01	
Traveling expense	583.03	
Purchase of Oakland County Highway Bonds	3,025.59	
Purchase of 3½% Treasury Certificates	2,001.88	
	$36,678.56	
Less cash discounts	93.34	
Total Disbursements		36,585.22
Balance in bank, per ledger, June 30, 1928		$7,273.17

AMERICAN CONCRETE INSTITUTE
BANK RECONCILIATION

June 30, 1928

Balance, per ledger, June 30, 1928	$7,273.17
Add checks outstanding	475.21
Balance per statement of National Bank of Commerce of Detroit, as at June 30, 1928	$7,748.38

LIST OF CONVENTION REGISTRANTS

FEBRUARY 12–14, 1929

* Members

*Abel, Norman, Allegheny County, D.P.W., Pittsburgh, Pa.
Abram, J. D., Pres. Abram Cement Tool Co., 2300 Michigan Ave., Detroit, Mich.
*Abrams, Duff A., 342 Madison Ave., New York, N. Y.
Adams, William H., 409 Hofman Bldg., Detroit, Mich.
Adelson, Louis, Eastern Cast Stone Co., 400 Eastern Ave., Malden, Mass.
*Ahlers, John G., Barney-Ahlers Corp., 110 W. 40th St., New York, N. Y.
*Albrecht, R. W., Plastic Products Co., 1991 Port Washington Rd., Milwaukee, Wis.
*Alexander, E. C., Massey Concrete Products Corp., 968 Peoples Gas Bldg., Chicago, Ill.
*Allan, W. D. M., Portland Cement Association, 33 W. Grand Ave., Chicago, Ill.
Allen, C. L., Michigan State College, 407 Grove St., East Lansing, Mich.
Allen, Harold J., Whithead & Kales, Detroit, Mich.
*Allen, H. R., The Economy Concrete Co., New Haven, Conn.
Allton, Robert A., 143 Sherman Ave., Columbus, Ohio.
Allyn, E. H., Paving Inspector, Detroit, Mich.
*Ames, Geo. M., Owen-Ames & Kimball Co., Grand Rapids, Mich.
*Anderegg, F. O., Mellon Institute, Pittsburgh, Pa.
Anderson, G. K., 5716 Cedar Ave., Philadelphia, Pa.
*Anderson, Louis, Alpha Portland Cement Co., Easton, Pa.
*Anthony, Walter B. E., 4862 Anderdon Ave., Detroit, Mich.
Arms, Leo M., Chicago Concrete Post Co., 4727 N. Lamon Ave., Chicago, Ill.
*Ashton, Ernest, Lehigh Portland Cement Co., Allentown, Pa.
*Ayres, Lewis E., Ann Arbor, Mich.
*Backemeyer, E. H., McCracken Pipe & Machinery Co., 50 Broad St., New York, N. Y.
Bagby, F. C., Western Waterproofing Co., Detroit, Mich.
*Bailey, Sam M., Wright Cast Stone, Inc., Jackson, Miss.
Baker, H. E., Wayne County Road Commission, Detroit, Mich.
*Baranzelli, G., Baranzelli Cast Stone, Inc., New York, N. Y.
*Bartlett, I. L., The Art Stone Co., Miller Falls, Mass.
*Bartram, Geo. C., 262 Walnut St., East Aurora, N. Y.
*Batchelder, Hudson Fireproof Brick Co., Boonton, N. J.
*Battyer, N. H., Lamar Pipe & Tile Co., Grand Rapids, Mich.
*Bauer, E. E., University of Illinois, Urbana, Ill.
*Bauman, E. W., Tennesee State Highway Dept., Nashville, Tenn.
*Bates, P. H., Bureau of Standards, Washington, D. C.
Beal, R. L., Huron Portland Cement Co., Wyandotte, Mich.
*Beer, F. E., Peerless Artificial Stone, Ltd., Toronto, Ontario, Canada.
Bendt, F. P., Koppers Construction Co., Pittsburgh, Pa.
Benjamin, O. R., The Detroit Edison Co., 8736 Quincy St., Detroit, Mich.
*Benson, Newton D., 36 Burrington St., Providence, R. I.
*Bertin, R. L., White Construction Co., Inc., 95 Madison Ave., New York, N. Y.
*Bespalow, E. F., Hollywood Concrete Pipe Co., Memphis, Tenn.
*Bigler, H. P., Rail Steel Bar Association, Chicago, Ill.

Bilodeau, William, Duntile Co., Ltd., 4626 Delaudiere St., Montreal, Quebec, Canada.
*Bischoff, John M., 4893 Courville St., Detroit, Mich.
*Bishop, Howard, Sharon Steel Hoop Co., 2317 Elm St., Youngstown, Ohio.
Bishop, Thomas W., Faithill, Ontario, Canada.
Boarts, Robert M., 2800 W. Grand Blvd., Detroit, Mich.
*Bogue, R. H., Bureau of Standards, Washington, D. C.
Bowman, Waldo G., *Engineering News-Record*, 36th St. and 10th Ave., New York, N. Y.
*Boyer, E. D., President American Concrete Institute, 25 Broadway, New York, N. Y.
*Bradler, C. M., The Youngstown Sheet & Tube Co., 237 W. 46th St., Gary, Ind.
Bradshaw, G. L., Independent Block & Cement Co., Indianapolis, Ind.
*Bragger, E. Y., Sandusky Cement Co., Shawomet, R. I.
*Brekke, G. N., American Bemberg Co., Johnson City, Tenn.
*Brickett, Edward M., Lehigh Portland Cement Co., Allentown, Pa.
Broander, E., The Aerocrete Corp. of America, 51 E. 42nd St., New York, N. Y.
*Brock, A. S., Aerocrete Western Corp., Chicago, Ill.
Brodine, A. R., 8047 Hamilton Ave., Detroit, Mich.
*Broker, A. E., Badger Cement Tile Co., Plymouth, Wis.
Brookman, Jr., Louis, *Concrete*, 139 N. Clark St., Chicago, Ill.
*Brown, F. E., Smith & Brown, 307 N. Michigan Ave., Chicago, Ill.
Browne, Herbert A., Wyandotte Portland Cement Co., Wyandotte, Mich.
Bruce, J. O., Hattiesburg, Miss.
*Bruce, W. C., American Tripoli Co., St. Louis, Mo.
Brueckner, M. G., Detroit Edison Co., 1944 Lothrop, Detroit, Mich.
*Buente, C. F., Concrete Products Co. of America, Pittsburgh, Pa.
*Bullen, C. H., Mid-West Concrete Pipe Co., 321 N. Central Ave., Chicago, Ill.
Burley, Fred H., Detroit Dept. of Water Supply, 590 Parkview, Detroit, Mich.
*Burks, I. E., Aluminum Co. of America, Kenogami, Quebec, Canada.
*Burns, Albert S., Asst. Chemical Engr., City of Indianapolis, Ind.
Burr, Floyd L., 718 Hammond Bldg., Detroit, Mich.
*Caldwell, F. E., Keystone Portland Cement Co., 4 Appleton Pl., Glen Ridge, N. J.
Caldwell, H., 2615 17th St., N. E., Washington, D. C.
Campbell, R. D., General Cement Products Corp., Pittsburgh, Pa.
*Campbell, R. H., 90 Wildwood Ave., Montclair, N. J.
*Campbell, S. A., Ridge Construction Corp., Rochester, N. Y.
*Canon, F. E., Pierson Concrete Products Co., 89 Dodd St., East Orange, N. J.
*Capouch, M. E., American Steel & Wire Co., Chicago, Ill.
*Carlson, C. A., Hawthorne Roofing Tile Co., Chicago, Ill.
Carton, A., Farnbank Block, Ltd., 208 Glenholme Ave., Toronto, Ontario, Canada.
*Carty, Bruce F., The Maul Co., Detroit, Mich.
*Chamier, A. U., Dept. of Water Supply, 8100 W. Warren Ave., Detroit, Mich.
*Chandler, Geo. D., Mt. Morris, N. Y.
*Chapman, Cloyd M., 105 W. 40th St., New York, N. Y.
Chill, Chas. W., Detroit Edison Co., 2000 Second Ave., Detroit, Mich.
*Chittenden, Howard L., Clinton, Conn.
*Christensen, Einar, 1600 Arch St., Philadelphia, Pa.
Christensen, J. C. V., Cincinnati Union Terminal Co., 1326 Edward Rd., Cincinnati, Ohio.
Christiansen, Detroit Dept. Water Supply, 14248 Coyle Ave., Detroit, Mich.
*Christoffel, Christoffel Art Stone Co., Milwaukee, Wis.
*Chubb, J. H., 131 46th St., New York, N. Y.
*Cissel, J. H., University of Michigan, Ann Arbor, Mich.
*Claye, Robert, Societes des Ciments Portland, Beaumont-sur-Oise, Sota, France.
*Clegg, Robert R., Havlik Stone Co., 414 Penobscot Bldg., Detroit, Mich.
*Clousing, Louis, 213 City Hall, Minneapolis, Minn.
*Coffman, Herbert, Hercules Cement Corp., Philadelphia, Pa.
Cohen, Sigmund, Dept. of Water Supply, 1560 Fairview, Detroit, Mich.
*Colburn, D. S., 140 S. Dearborn St., Chicago, Ill.
*Collins, D. R., 47th and State Sts., Milwaukee, Wis.
*Collins, H. G., Lehigh Portland Cement Co., Columbus, Ohio.
*Collins, James G., Lockstrip Mfg. Corp., 175 116th St., New York, N. Y.

*COLMAR, DANIEL, Ramloc Stone Co., Inc., 1055 Broadway, Albany, N. Y.
*CONAHEY, GEORGE, The Celite Corp., 1824 "S" St., N. W., Washington, D. C.
CONNER, V. N., Universal Portland Cement Co., Pittsburgh, Pa.
*CONROW, A. D., Ash Grove Lime & Portland Cement Co., Oklahoma City, Okla.
*CRABBS, AUSTIN, The Cement Products Co., Davenport, Iowa.
CRAPO, WILLIAM W., Huron Portland Cement Co., 2204 Seminole Ave., Detroit, Mich.
*CROSS, HARDY, University of Illinois, Urbana, Ill.
CROSBY, E. S., Johns-Manville, Maplewood, N. J.
*CRUM, R. W., National Research Council, Washington, D. C.
*CRUME, W. H., 511 Keith Bldg., Dayton, Ohio.
CUNNINGHAM, GEORGE B., Detroit Edison Co., 599 St. Clair Ave., Detroit, Mich.
CUSACK, FRANK, Detroit Edison Co., 75 Crawford Ave., Windsor, Ontario, Canada.
CUTLER, S. G., Olney J. Dean Co., 137 S. LaSalle St., Chicago, Ill.
DAIR, CHARLES W., Bessemer Cement Corp., 518 W. Sandusky St., Findley, Ohio.
DAVIDSON, A. J., The Detroit Edison Co., Detroit, Mich.
*DAWSON, JAMES H., Portland Cement Association, 13291 Freeland Ave., Detroit, Mich.
DEAN, C. L., Southwestern Portland Cement Co., Osborn, Ohio.
*DEIGNAN, J. E., Tamper & Equipment Co., 80 E. Jackson Blvd., Chicago, Ill.
*DEINBOLL, F. K., New York Central R. R., Cleveland, Ohio.
*DELAMATER, C. S., Consolidated Concrete Machy. Corp., Adrian, Mich.
*DELEVANTE, C. A., Mohawk Stucco Co., Inc., Brooklyn, N. Y.
*DERLETH, C. P., Colloy Products Co., 4030 Chouteau St., St. Louis, Mo.
DESTEFANO, ANTHONY, 1530 80th St., New York, N. Y.
*DEVER, CLARE, 1325 Ford Bldg., Detroit, Mich.
DEVEREAUX, P. J., Fairbank Block, Ltd., Toronto, Ontario, Canada.
*DEVLIN, JOHN J., 100 E. 42nd St., New York, N. Y.
DICKINSON, C. H., Detroit Edison Co., 2000 Second Ave., Detroit, Mich.
*DIEHL, KENT, Ash Grove Lime Cement Co., Kansas City, Mo.
*DIENHART, E. W., Box 1116, Cleveland, Ohio.
DILLMAN, G. E., State Highway Commission, Lansing, Mich.
*DOE, NELSON L., 362 Egmont Ave., Mt. Vernon, N. Y.
DOLAND, J. J., 604 W. Nevada, Urbana, Ill.
*DONOVAN, JOHN B., Jackson, Mich.
DORSEY, J. W., Dorsey Supply Co., 8545 Dumbarton Rd., Detroit, Mich.
*DOUGLAS, M. S., Case School of Applied Science, Cleveland, Ohio.
*DOUGLASS, A. S., The Detroit Edison Co., Detroit, Mich.
DOWDALL, E. J., Universal Portland Cement Co., 208 S. LaSalle St., Chicago, Ill.
*DOWNER, ROBT. J., Irish Hill Supply Co., Runnemede, N. J.
*DOWNS, C. S., Artstone Products, Inc., 52 Vanderbilt Ave., New York, N. Y.
*DRIESBACH, EDW. E., Standard Lime & Stone Co., Martinsburg, W. Va.
DRITTLER, CARL F., Detroit Edison Co., 1049 Lenox Ave., Detroit, Mich.
*DUNNELLS, C. G., Century Bldg., Pittsburgh, Pa.
*DUTTON, EARL, Tredennik & Billings Co., 10 High St., Boston, Mass.
*DYESS, J. W., Brick Tile & Steel Form Co., 137 W. 5th St., Hattiesburg, Miss.
*EASTERDAY, PHIL, Terre Haute, Ind.
*EDDY, PAUL E., Onondaga Litholite Co., Syracuse, N. Y.
*EDWARDS, C. E., Lamar Pipe & Tile Co., 538 Morris Ave., Grand Rapids, Mich.
EIDE, TORRIS, Dept. of Water Supply, Water Works Park, Detroit, Mich.
*EKHOLM, S. L., The MacGlashan Corp., Tecumseh, Mich.
*ELKAN, STANLEY, President Elkan Stone-Tile Mfg. Co., Macon, Ga.
*EMERSON, H. B., Lehigh Portland Cement Co., 111 W. Washington St., Chicago, Ill.
*EMERSON, F. M., Emerson & Norris Co., Swampscott, Mass.
*EMERY, W. C., Consolidated Concrete Machy. Corp., Adrian, Mich.
*EMMONS, W. J., Michigan State Highway Laboratory, Electrical Engineering Bldg., Ann Arbor, Mich.
*ENGHOLM, F. G., 321 Bloor Bldg., Toronto 5, Ontario, Canada.
*EPPES, N. A., Gulf Concrete Pipe Co., Houston, Tex.
ERICKSON, J. A., 531 Dorothy Lane, East Lansing, Mich.
*EVANS, J. CLIFFORD, Nazareth Cement Co., Nazareth, Pa.

EVERETT, ROY E., Stevens & Wood, Inc., Jackson, Mich.
*EYRICK, GEO. J., Smith, Hinchman & Grylls, Detroit, Mich.
*FAIRCHILD, L. F., 3195 Lake Ave., Rochester, N. Y.
*FALCO, L. A., The Decorative Stone Co., New Haven, Conn.
*FARMER, HOMER G., Universal Portland Cement Co., Chicago, Ill.
*FATOUT, R. T., Hall Construction Co., 406 Board of Trade, Indianapolis, Ind.
*FAVORITE, WM. P., Blaw-Knox Co., 2908 W. Grand Blvd., Detroit, Mich.
FEATHER, PRESTIN, Petoskey, Mich.
*FEDDELER, MELVIN E., 228 N. LaSalle St., Chicago, Ill.
FREDERICI, ALBERT, 300 E. Railway Ave., Paterson, N. J.
FELLOWS, PERRY A., 1364 Newport Ave., Detroit, Mich.
*FENKELL, GEO. H., Gen. Mgr. Dept. of Water Supply, Detroit, Mich.
FENKELL, NEAL H., 9591 Ohio Ave., Detroit, Mich.
*FERRER, RUSSELL R., Center Moriches, Long Island, N. Y.
FERTIG, GEORGE J., 4301 Cliff Road, Birmingham, Ala.
FISHER, MARTIN R., 2684 Carter Ave., Detroit, Mich.
*FITCH, H. M., Rock Products, Pontiac Bldg., Chicago, Ill.
FLEMING, E. M., Portland Cement Assn., 33 W. Grand Ave., Chicago, Ill.
*FLYNN, THOS. P., 6153 Wabash Ave., Detroit, Mich.
*FOLEY, R. A., Superior Products Co., Detroit, Mich.
*FORMIGLI, O. L., Formigli Arch Stone Co., Philadelphia, Pa.
FORSBERG, GEORGE W., Forsberg Cement Block Co., Worcester, Mass.
*FOSTER, ALEXANDER, Chas. Warner Co., 1518 Walnut St., Philadelphia, Pa.
*FOSTER, C. B., 726 K. of P. Bldg., Indianapolis, Ind.
FOUNTAIN, GEORGE F., Port Huron, Mich.
*FRAUENFELDER, HERMAN, 2024 Fair Ave., St. Louis, Mo.
*FRANKLIN, JACK, Wisconsin Concrete Products Assn., Milwaukee, Wis.
FRAYLICK, ARTHUR M., Detroit Edison Co., Detroit, Mich.
FREEMAN, FRANK B., 5737 Williamson, Fordson, Mich.
*FREEMAN, P. J., 519 Smithfield St., Pittsburgh, Pa.
*FRIBERG, BENJ., Laclede Steel Co., St. Louis, Mo.
*FRIIS, KRISTEN, Munkedamsvejen 3, Oslo, Norway.
FRIIS, NELSON A., Architectural Stone Co., 1118 E. 77th St., Chicago, Ill.
*FULCHER, EDMUND, 404 22nd St., Brandon, Manitoba, Canada.
*FULLENWIDER, C. V. R., The Philip Carey Co., Cincinnati, Ohio.
GARLOCK, C. B., 2023 Tuxedo St., Detroit, Mich.
GARRITY, LEO V., 3265 Boston Blvd., Detroit, Mich.
GAY, MAURICE, Detroit Edison Co., 8710 Dumbarton Rd., Detroit, Mich.
GILBERT, L. C., France Stone Co., 4042 Wetzler Rd., Toledo, Ohio.
GILMORE, DAVID P., Prof. of Civil Engineering, University of Detroit, Detroit, Mich.
*GINSBERG, FRANK I., H. O. Penn Co., 18 E. 41st St., New York, N. Y.
*GOLDBECK, A. T., Nat'l Crushed Stone Association, Washington, D. C.
*GOODING, O. E., Standard Gravel Co., New Hudson, Mich.
*GONNERMAN, H. F., 33 W. Grand Ave., Chicago, Ill.
*GORSUCH, V. D., Bessemer Cement Corp., Youngstown, Ohio.
GOSSWEIN, O. H., Universal Portland Cement Co., 210 S. LaSalle St., Chicago, Ill.
*GOULD, H. J., Ferro Concrete Construction Co., Cincinnati, Ohio.
*GRAM, LEWIS M., Ann Arbor, Mich.
GRASSMAN, H. S., 4214 Three Mile Drive, Detroit, Mich.
GREEN, EDWIN G., 17611 Ohio Ave., Detroit, Mich.
GREENBERG, JOSEPH, 718 Hammond Bldg., Detroit, Mich.
GREY, ALLEN J., 14550 Mettetal Ave., Detroit, Mich.
*GRIFFITH, E. A., 410 Hampton Ave., Wilkinsburg, Pa.
GROBBEL, D. C., Dept. Water Supply, Detroit, Mich.
GROESBECK, R. C., American Steel & Wire Co., Detroit, Mich.
GUNNERSON, HUGH B., Detroit Edison Co., Detroit, Mich.
GUY, FRANK E., Universal Portland Cement Co., 518 Frick Bldg., Pittsburgh, Pa.
*HAEGERT, LEE V., 2240 W. 8th St., Topeka, Kan.
HAGEN, WILLIAM A., Detroit Public Works, 933 Adeline Ave., Detroit, Mich.

*Haight, E. E., Concrete Publishing Co., 139 N. Clark St., Chicago, Ill.
Hallihan, John P., 7409 2nd Blvd., Detroit, Mich.
Hammill, Harold F., 112 Union St., Plymouth, Mich.
Hamley, S. J., Forest City Testing Laboratory, Cleveland, Ohio.
Hand, F. A., Supt. Kalman Floor Co., 7616 Byron St., Detroit, Mich.
*Hanley, Lance, Portland Cement Association, Detroit, Mich.
*Hardesty, J. M., 463 West St., New York, N. Y.
Hardin, Eugene A., 3347 Boston Blvd., Detroit, Mich.
Hardy, John R., Dept. of Water Supply, Detroit, Mich.
Hare, E. Harold, 4547 Wentworth Ave., Detroit, Mich.
Harper, E. W., Ontario Cement Products Co., Toronto, Ontario, Canada.
*Harris, Wallace R., Plaza Hotel, Milwaukee, Wis.
*Hart, W. E., Portland Cement Assn., 33 W. Grand Ave., Chicago, Ill.
Hart, Thos. A., 27121 Oak Dale Ave., Royal Oak, Mich.
*Harter, B. R., Harter-Marblecrete Stone Co., Oklahoma City, Okla.
Harvey, B. W., Mt. Clemens, Mich.
Hastings, Carl A., Hash Concrete Products, Niagara Falls, N. Y.
*Hatt, W. K., Purdue University, Lafayette, Ind.
Havill, C. H., Superior Products Co., Transportation Bldg., Detroit, Mich.
*Hayden, J. E., Crystalite Reduction Co., Ishpeming, Mich.
*Healy, J. T., The Pumicite Co., 1914 Builders Bldg., Chicago, Ill.
*Hedderich, H. F., 3850 Bingham Ave., St. Louis, Mo.
*Heiby, W. H., Marietta Concrete Corp., Marietta, Ohio.
Hermann, Karl, 13914 Young St., Detroit, Mich.
*Henderson, J. H., Granite Concrete Block Co., Ltd., Toronto, Ontario, Canada.
Hennes, Robert G., 1721 McClellan Ave., Detroit, Mich.
*Herzog, C. J., General Cement Products Co., 310 Magee Bldg., Pittsburgh, Pa.
*Hickman, A. S., W. E. & M. Co., East Pittsburgh, Pa.
Highland, Elmer, Detroit Water Supply, 2804 6th St., Detroit, Mich.
*Hinchman, T. H., Smith, Hinchman & Grylls, Marquette Bldg., Detroit, Mich.
*Hirschberg, W. P., 218 Stephenson Bldg., Milwaukee, Wis.
Hoffman, J. C., Portland Cement Assn., Dime Bank Bldg., Detroit, Mich.
*Hoffman, Martin, Detroit Cast Stone Co., 8506 South St., Detroit, Mich.
Holcomb, A. E., Koehring Co., Milwaukee, Wis.
Hollingworth, W., Ellicott Square, Buffalo, N. Y.
Housel, William S., University of Michigan, 1119 Spring St., Ann Arbor, Mich.
*Howe, Henry L., Dept. of Engineering, City of Rochester, N. Y.
*Hubbard, Fred, Youngstown, Ohio.
Huff, C. B., Eastern Clay Products Assn., 1040 Free Press Bldg., Detroit, Mich.
Hughes, H. Walter, 14 Court St., Rochester, N. Y.
Hunter, Ernest L., Chief, Division of Drains, Charlotte, Mich.
*Huntley, N. H., 2406 Richton Ave., Detroit, Mich.
*Hussey, Jack W., Atlas Portland Cement Co., St. Louis, Mo.
*Hutchins, L. A., Wolverine Portland Cement Co., Coldwater, Mich.
Imler, Roy F., Concrete Products Magazine, 53 W. Jackson Blvd., Chicago, Ill.
Jackson, R. W., 14035 Mark Twain Ave., Detroit, Mich.
*Jackson, Frank H., U. S. Bureau of Public Roads, Washington, D. C.
*Jenks, C. T., Lamar Pipe & Title Co., Grand Rapids, Mich.
Jennings, Dwight F., Huron Portland Cement Co., Ford Bldg., Detroit, Mich.
*Jennings, W. A., Southwestern Portland Cement Co., Osborn, Ohio.
Jerz, Walter John, Dept. Water Supply, 3352 Grand Ave., Detroit, Mich.
*Jackson, C., Electric Tamper Equipment Co., 80 E. Jackson Blvd., Chicago, Ill.
*Johns, H. S., Crystalite Stone Products Co., Hillyard St., Hamilton, Ontario, Canada.
Johnson, R. P., Detroit Edison Co., 972 Beaconsfield Ave., Grosse Pointe, Mich.
*Johnson, N. C., 325 Madison Ave., New York, N. Y.
*Johnson, R. W., Concrete Reinforcing Steel Inst., Chicago, Ill.
Johnston, Clair C., University of Detroit, 1103 Seyburn Ave., Detroit, Mich.
*Johnston, Hugh H., South Dakota Concrete Products Co., Watertown, S. Dak.
*Johnston, W. J., St. John, New Brunswick, Canada.

JONAS, JOSEPH F., The Detroit Edison Co., 718 Pleasant St., S., Royal Oak, Mich.
*JONES, H. C., Huron Portland Cement Co., Detroit, Mich.
*JONES, C. L., Havlik Stone Co., Chicago Heights, Ill.
JONES, EVERETT L., 141 Massachusetts, Detroit, Mich.
KALLGREN, H. R., Dept. of Water Supply, 2578 Harding Ave., Detroit, Mich.
KANTOLA, WILLIAM V., 2317 Dime Bank Bldg., Detroit, Mich.
*KAUFMAN, R. R., The Master Builders Co., 3548 W. 99th St., Cleveland, Ohio.
KAYSER, CHARLES B., Cleveland, Ohio.
*KEAN, R. E., Superior Products Co., Transportation Bldg., Detroit, Mich.
*KEARNEY, JAMES C., Western Waterproofing Co., 1714 Penobscot Bldg., Detroit, Mich.
*KEATINGE, PAUL F., Atlas Portland Cement Co., St. Louis, Mo.
*KENNEDY, JOHN W., Huron Portland Cement Co., Ford Bldg., Detroit, Mich.
KIMMICH, FREDERICK A., Detroit Edison Co., 2519 Seminole Ave., Detroit, Mich.
KLENK, FRED, Forest City Testing Laboratory, Cleveland, Ohio.
*KREHBIEL, B. F., Cement Stone & Supply Co., Wichita, Kan.
KRUEGER, HARRY, Detroit Water Board, 3836 Crane Ave., Detroit, Mich.
KROELL, JOHN T., Dept. of Water Supply, 6334 Alaska Ave., Detroit, Mich.
KRUEGER, JULIUS A., The Toledo Cast & Art Stone Co., Toledo, Ohio.
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